Robert R. Marriam, Consultant
Remedium Group, Inc.
A Subsidiary of W. R. Grace & Co.
6401 Poplar Ave., Suite 301
Memphis, TN 38119
Tel: (901) 820-2023
Fax: (901) 820-2061

Remedium

June 27, 2008

Ms. Bonita Lavelle
US EPA Region 8
EPR - SR
1595 Wynkoop Street
Denver, CO 80202-1129

Dear Bonnie,

Please find enclosed the following reports:

1. Geotechnical Evaluation, W R Grace Dam, Rainy Creek, Montana

2. Engineering Analysis of Fllod Routing Alternatives for the W R Grace Vermiculite Tailings Impoundment

Please advise if there are any questions.

Robert R. Marriam

Cc: Catherine LeCours, Montana DEQ
W. M. Corcoran (w/out attachments)
R. J. Medler (w/out attachments)

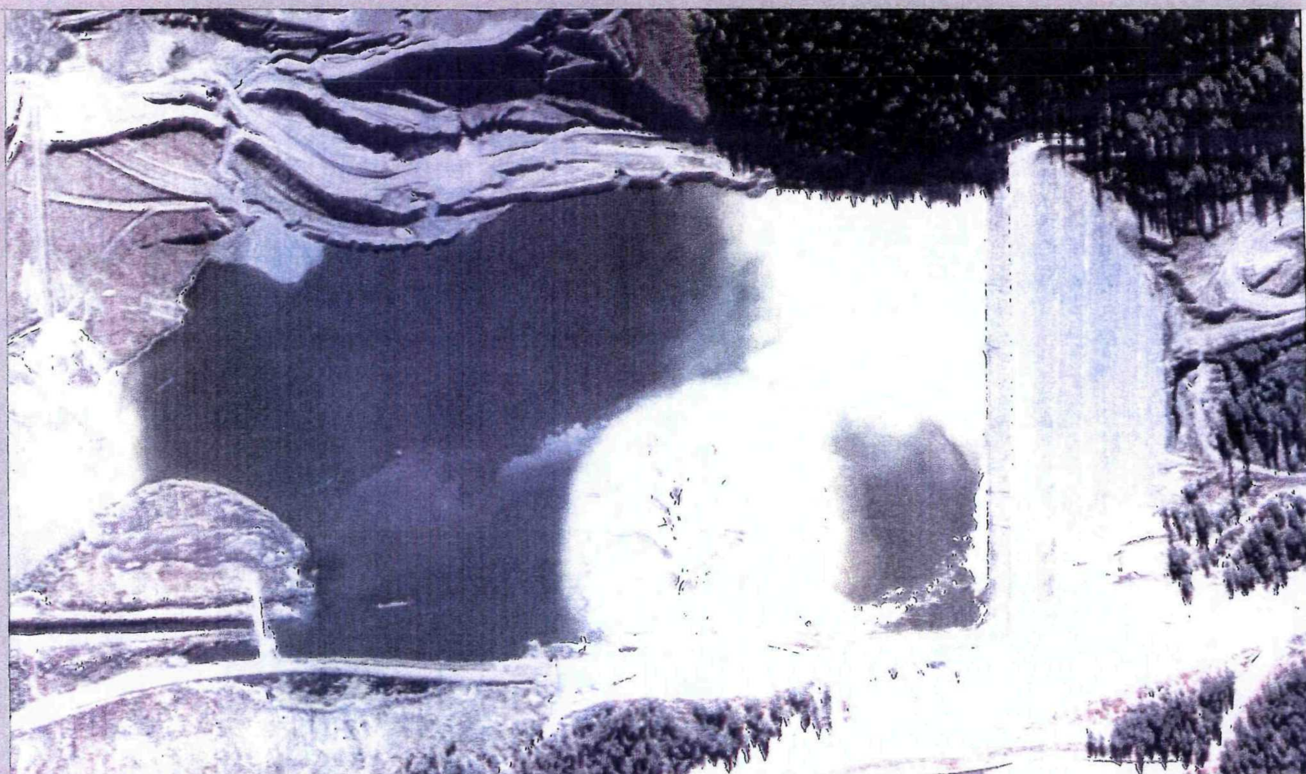
dwp
Enclosures

FINAL REPORT

ENGINEERING ANALYSIS OF FLOOD ROUTING ALTERNATIVES

for the

W.R. GRACE VERMICULITE TAILINGS IMPOUNDMENT

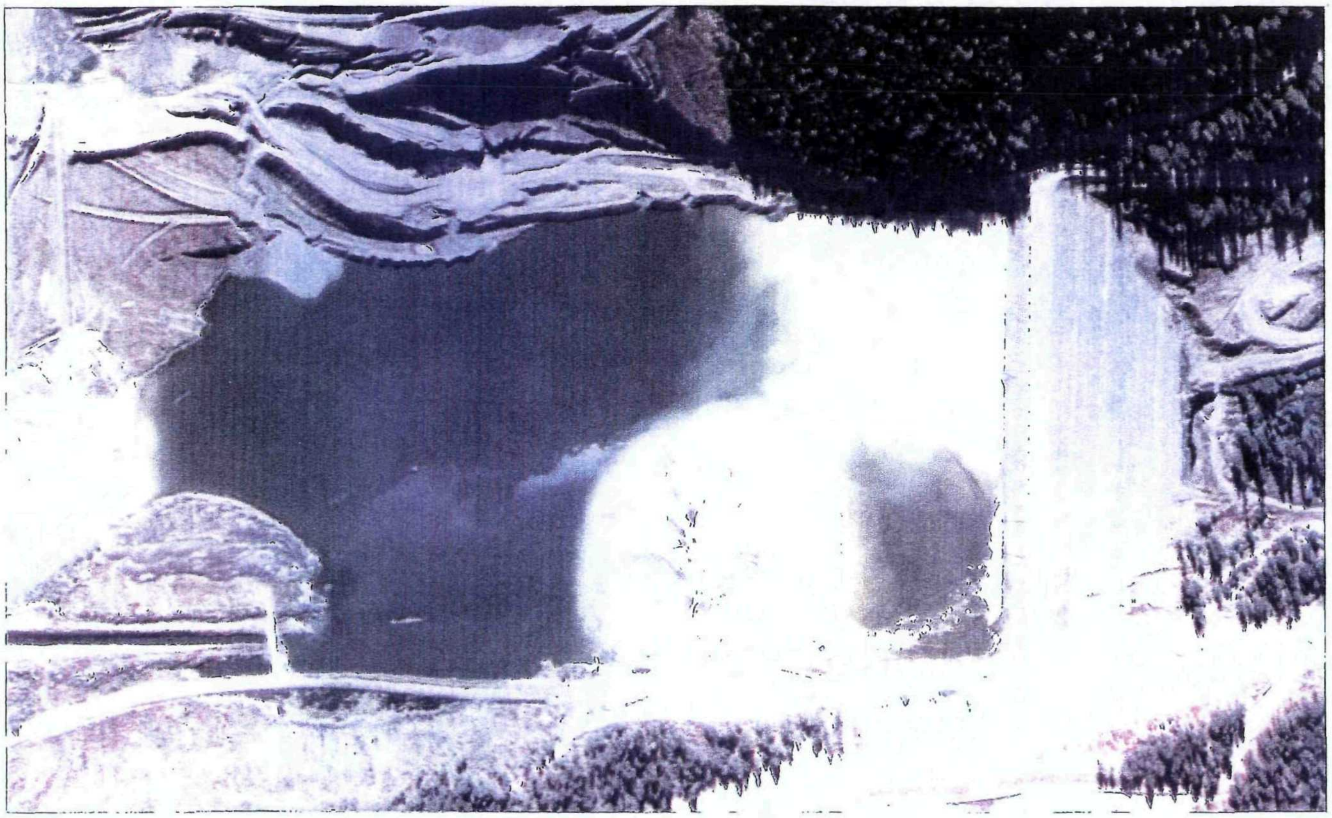


SUBMITTED TO:
STATE OF MONTANA
DEPT. OF STATE LANDS
HELENA, MONTANA

SUBMITTED BY:
SCHAFER AND ASSOCIATES
P.O. BOX 6186
BOZEMAN, MONTANA

March, 1992

ENGINEERING ANALYSIS OF FLOOD ROUTING ALTERNATIVES
for the
W.R. GRACE VERMICULITE TAILINGS IMPOUNDMENT LIBBY, MONTANA



by:

Bruce K. Parker, P.E.[1]

Thomas J. Hudson[2]

1 - Manager of Engineering Services, Schafer and Associates, Bozeman, MT
2 - Engineering Group Leader, Schafer and Associates, Bozeman, MT

LSB 66 00002

Table of Contents

LSB 66 00004

List of Tables

List of Figures

List of Drawing Plates

Appendices

Number	Description
Appendix A	Hydrologic Modeling Results
Appendix B	Probable Maximum Flood Calculations
Appendix C	Control Structure and Emergency Spillway Calculations
Appendix D	Flood Routing Results
Appendix E	Standard Drawing-SCS Drop Structure

1.0 EXECUTIVE SUMMARY

An engineering analysis of flood routing alternatives was completed at the W.R. Grace vermiculite tailings impoundment, near Libby, Montana, to investigate the various alternatives for routing floods through the tailings impoundment following closure. W.R. Grace has ceased mining and milling operations at the site and wishes to complete closure operations and requirements during 1992 in order to obtain bond release.

Regulatory agencies, including the Department of State Lands (DSL), USDA Forest Service, and others have raised concerns over the mine closure, particularly the closure of the tailings impoundment. These concerns include:

- asbestiform fiber contamination in surface water from the coarse tailings dump and fine tailings impoundment;
- long-term stability and integrity of the dam, primarily with regard to saturation and seepage failure;
- increased sedimentation of downstream areas from the impoundment;
- safety; and finally,
- setting a precedence for other tailings impoundments.

In order to address these issues, an engineering analysis of flood routing alternatives was conducted. The purpose of the engineering analysis was to objectively examine the various alternatives for routing Rainy Creek and Fleetwood Creek flows through the area affected by the vermiculite tailings impoundment, and to present a conceptual plan of the preferred alternative. The analysis addressed the issues of hydrology and flood routing, dam safety, short-term and long-term environmental impact, construction feasibility, costs, long-term stability and erosion control, and proposed reclamation methods and practices.

The impoundment is situated on Rainy Creek, immediately below the confluence with Fleetwood Creek, and impounds approximately 9.4 square miles of the Rainy Creek drainage area. A design flood of 0.5 PMF, calculated at 5838 cfs, was selected as the inflow volume that would be used for flood routing through the impoundment.

The investigation determined that the best method to safely pass a design storm of this magnitude in a stable manner, while assuring the long-term integrity of the dam, is to route the storm <u>through</u> the impoundment using controlled outflow structures. By using the impoundment to temporarily store peak inflows, outflow volumes can be reduced to a fraction of the 0.5 PMF peak inflow volume.

Routing the floods through the impoundment using controlled outflow structures provided the safest and most cost effective method of flood routing for the tailings impoundment while addressing the majority of the regulatory concerns. Significant advantages include:

- Provides a higher level of public safety than other alternatives while assuring the long-term integrity of the tailings dam and retaining a relatively straightforward design;

Combined

- Provides a cost-effective, relatively straightforward method of safely handling storm flows;
- During a 0.5 PMF event this design is geotechnically the most stable of the alternatives;
- System is capable of handling floods larger than the design flood of 0.5 PMF with the addition of an emergency spillway;
- Outflows are considerably less than 0.5 PMF due to flood routing, allowing for a smaller, more cost effective channel, and less downstream disturbance during major events;
- Environmental disturbance is kept to a minimum with the a smaller, more natural outflow channel;
- The remaining impoundment wetland promotes surface water improvement through natural filtration and settlement;
- Least overall maintenance of the alternatives;
- Minimal water loss to infiltration; and,
- Impoundment wetland would provide excellent wildlife habitat.

2.0 INTRODUCTION

2.1 OVERVIEW/PROJECT OBJECTIVES

W.R. Grace and Company, Zonolite Division, Libby, Montana, has retained Schafer and Associates, Bozeman, Montana, to perform an Engineering Analysis of Flood Routing Alternatives for Rainy Creek and Fleetwood Creek, which have been affected by a vermiculite tailings impoundment. The impoundment was constructed to provide process water and settle tailings at W.R. Grace's vermiculite mining/milling operations northeast of Libby. Currently, Rainy Creek is intercepted above the impoundment, and diverted around the tailings impoundment through a culvert constructed of 48 and 52 inch diameter corrugated metal pipe, re-entering the original channel below the tailings dam. Fleetwood Creek enters the impoundment through a constructed diversion channel.

W.R. Grace has ceased operations at the entire mining, milling, and shipping facilities, and has begun implementing reclamation and closure measures at the site. It is the desire of W.R. Grace to complete all reclamation and closure requirements during 1992, and obtain bond release for the entire project area and facilities, including the tailings impoundment.

Regulatory agencies, including the Department of State Lands (DSL), USDA Forest Service, and others have raised concerns over the mine closure, particularly the closure of the tailings impoundment. These concerns include:

- asbestiform fiber contamination in surface water from the coarse tailings dump and fine tailings impoundment;

- long-term stability and integrity of the dam, primarily with regards to saturation and seepage failure;

- increased sedimentation of downstream areas from the impoundment;

- safety; and,

- setting a precedence for other tailings impoundments.

In order to address these issues, an engineering analysis of flood routing alternatives was conducted. The objectives of the engineering analysis are to examine the various alternatives for routing Rainy Creek and Fleetwood Creek flows through the area affected by the vermiculite tailings impoundment, and to present a conceptual plan of the preferred alternative. The analysis will address the issues of hydrology and flood routing, dam safety, environmental disturbance, construction feasibility, costs, long-term stability, erosion control, and proposed reclamation methods and practices. (Note: the issues of water quality and tailings dam stability are addressed in separate investigations titled "W.R. Grace Vermiculite Mine Closure Water Quality Monitoring Plan" (Hudson, 1991) and "Geotechnical Evaluation, W.R. Grace Dam, Rainy Creek, Montana" (Vahdani, 1992) respectively.

Various alternatives for collecting and routing Rainy and Fleetwood Creeks around or through the impoundment will be reviewed, with advantages and disadvantages considered and discussed. The ultimate objective is to provide a method of passing storm flows through the impoundment area assuring the integrity of the dam without producing significant environmental impacts in the form of water quality degradation or disturbances to local terrain.

- Our approach to meeting this objective is as follows:
- First, select suitable storm events which will be used as design criteria, determine size, and calculate runoff volumes for these storms (Chapter 3),
- Second, define and compare conceptual approaches and select a preferred alternative for detailed description (Chapter 4),
- Third, define essential elements of design for the preferred alternative and discuss possible alternatives for implementing details of design (Chapter 5),
- Finally, propose maintenance procedures which will be implemented to provide for the perpetual safety of the implemented closure plan (Chapter 6),

2.2 PROJECT LOCATION AND DESCRIPTION

The vermiculite tailings impoundment is part of W.R. Grace's Construction Products Division vermiculite operations. The tailings impoundment encompasses approximately 70 acres within the drainage basin(s) of Rainy and Fleetwood Creeks. The site is located approximately seven miles east northeast of Libby, Montana, within the SW 1/4 of Section 15, and the NW 1/4 of Section 22, Township 31 North, Range 30 West, Lincoln County, Montana. The site is accessed by State Highway 37, and USFS Road No. 401. The impoundment lies entirely within patented mine property owned by W.R. Grace and Company. Surrounding public land is managed by the USDA Forest Service, Libby Ranger District. See Figures 2.1 and 2.2.

The tailings impoundment is located immediately below the confluence of Rainy Creek and Fleetwood Creek. After leaving the mine property, Rainy Creek flows toward the southwest and enters the Kootenai River about 2 1/2 miles downstream of the dam, and about 5 1/2 miles upstream of Libby. The Kootenai River is a tributary of the Clark Fork

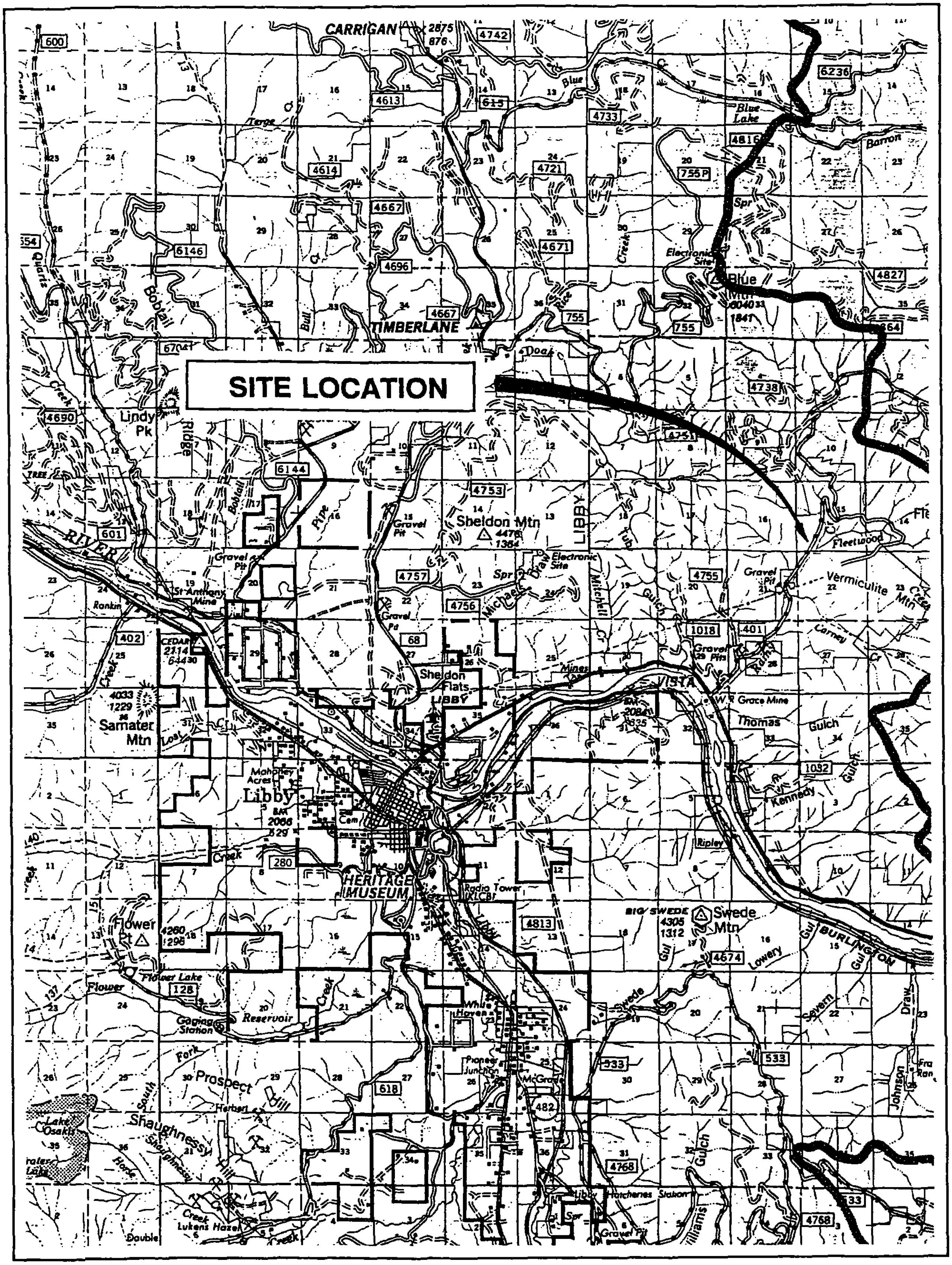


Figure 2.1 Location of the W.R. Grace Project Area.

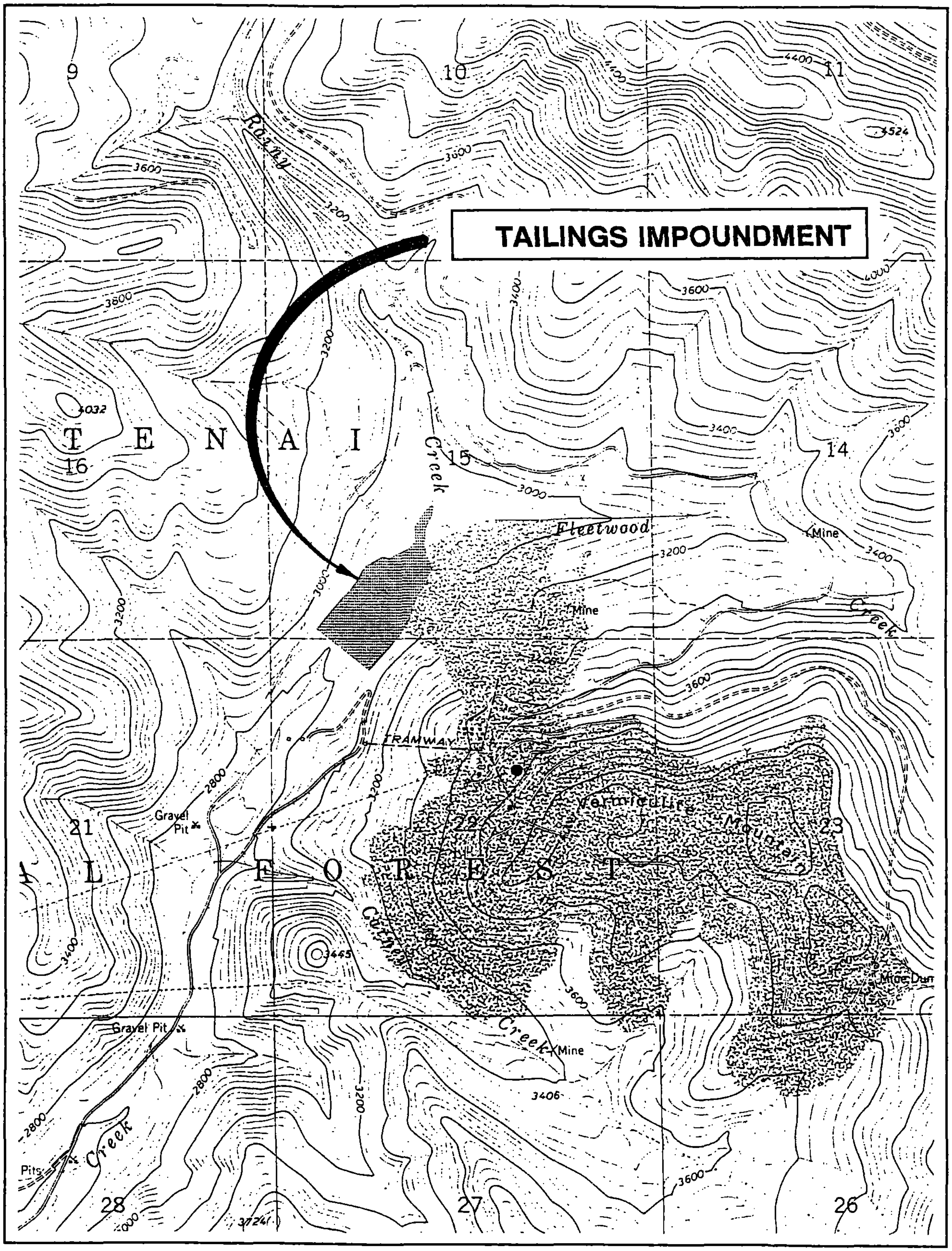


Figure 2.2 W. R. Grace Vermiculite Tailings Impoundment. USGS Vermiculite Mountain, Mont Quadrangle, Lincoln Co.

of the Columbia River. The total drainage area impounded by the tailings dam is a 9.4 square miles. The dam is rated as large in size, and is classified as having a high (Category 1) downstream hazard potential (Foster, 1981). The high hazard ranking is attributed to the presence downstream of Highway 37 and the vermiculite product storage and shipping terminal located between the highway and the Kootenai River.

Existing outlets from the impoundment consist of a decant tower and a chute spillway constructed of half-sections of 8 foot diameter corrugated metal pipe (CMP). Normal flows from Rainy Creek are currently diverted around the impoundment through a CMP pipe constructed of 48 and 52 inch diameter sections, re-entering the original channel approximately 800 feet downstream of the dam. All existing outlet and diversion structures will be removed as part of final closure.

The geology of the site consists of late Precambrian Belt Group consisting of fine-grained clastic and carbonate rocks which have undergone various degrees of metamorphism, and are covered with glacial outwash and till (Boettcher, 1963). The tailings impoundment is located on an intrusive rock body called the Rainy Creek stock, of which Vermiculite Mountain and W.R. Grace's mining area is a part. Depths to bedrock range from less than 2 feet to about 25 feet on the valley walls, and from 20 to 45 feet on the valley floor. Portions of the bedrock are weathered with low strength (Lewis, 1971).

The dam is located in Seismic Zone 2, with a potential for moderate earthquake damage. A study completed by Harding Lawson Associates (Vahdani, 1992) indicates "*....the dam is expected to remain stable during and following the design earthquake*", and "*...... results of our stability analysis indicate that the dam is stable during both static and dynamic loading conditions*".

Vegetation at the site consists of grasses, coniferous shrubs, and of mixture of deciduous (primarily cottonwood, alder, and aspen) and evergreen trees (cedar, larch, Douglas fir, ponderosa and lodgepole pine, and spruce). Active logging is taking place within the drainage basin, both on mine property and on adjacent Forest Service land. The tailings impoundment is currently devoid of vegetation.

2.3 SITE HISTORY/BACKGROUND

Vermiculite Mountain has long been the subject of mineral exploration because of the unique geology of the area. However, vermiculite production has been the only economically viable operation there. Mining was done as early 1890 but the first large scale activity was begun by the Zonolite Company beginning in the mid 1920's. W. R. Grace acquired the Zonolite Company in 1963 which continued to operate as the Zonolite Division of W.R. Grace. The first beneficiation process used an air separation method to process ore into a high grade vermiculite product. This process tended to produce high dust levels which took on increased significance with the recognition that asbestiform fibers could lead to certain kinds of lung disease. The ore body has occurrences of tremolite which is classified as an asbestos-like mineral. The process was converted to a wet process to reduce dust production during processing.

In 1971 W. R. Grace undertook a major expansion to increase capacity and improve the beneficiation process. It was at this time that the tailings impoundment was built to provide for settlement of the fine tails produced by the new process and to recover water for reuse (Foster, 1981; Boettcher, 1963; and Lewis, 1971). The tailings dam was designed by Bovay Engineers, Inc. of Spokane, Washington, and Harding Lawson Associates of Novato, California. The dam was designed and constructed in stages, with the 50 foot high (elevation 2830) starter dam constructed in 1971, immediately downstream of an older, existing dam. Additional construction phases in 1975, 1977, and 1980 have raised the top of dam elevation to 2925, for a total height of 135 feet measured from the downstream toe.

At the peak of operations, ore was processed at the rate of approximately 2,000,000 tons per year. Declining market conditions forced a gradual reduction in plant production from over 200,000 tons per year of product to less than 100,000 tons per year recently. In the fall of 1990 a decision was made to permanently close the facility because of the declining markets. Since 1990, the tailings impoundment has not received fine tails directly from the operations. However, small amounts of tailings from adjacent coarse tailings disposal areas continue to enter the reservoir through natural erosion processes, primarily surface runoff. These processes will be reduced as reclamation and reseeding efforts provide surface cover and stabilize the area.

A reclamation plan was submitted at the time of the expansion. However, the plan was very general and did not define or investigate specific actions in detail. One of the provisions of the permit was to provide for diversion of streams around mining wastes at the time of closure. In the case of the tailings impoundment, the requirements for diversion of a massive storm is calculated to be several thousand cubic feet per second. Our investigation of designs for successfully handling such a large quantity of water has suggested that other alternatives, using the storage capacity of the tailings impoundment might provide a safer and more effective resolution of this problem. The reasons for this conclusion are discussed in the sections which follow.

3.0 HYDROLOGIC EVALUATION

3.1 HYDROLOGIC PARAMETERS

In order to properly assess the requirements of the final closure design for the tailings impoundment it is necessary to evaluate the magnitude of streamflows for various levels of probability. We have analyzed three storm events here. A 10 year thunderstorm event was chosen to represent a condition which might be encountered on a regular basis and which might also be considered as a design parameter for some diversion alternates. A 100 year thunderstorm event was selected principally as the preferred basis for design of a partial diversion alternate, an event which would be exceeded only rarely thereby requiring use of emergency provisions on an infrequent time interval. A runoff equivalent to 0.5 of the probable maximum flood (PMF) event was also selected since the requirements for dam safety are based on the PMF and this value met or exceeded those requirements. There is also a recorded event in the area of a 0.5 PMF event. This event was a three day general storm; our analysis is based on a 6 hour thunderstorm event which produces a more intense runoff in a drainage of this size. The methodology for calculation of these design storms is described in Section 3.2.

The W.R. Grace tailings dam is located on Rainy Creek, approximately 2000 feet below the confluence of Rainy and Fleetwood Creeks. The dam impounds 9.4 square miles (sq. mi.) of the Rainy Creek drainage basin, of which 5.9 sq. mi. is drained by Rainy Creek, and 3.5 sq. mi. is drained by Fleetwood Creek. The two flows enter the impoundment from the north and east, respectively. The drainage basin is generally "L" shaped above the dam (Figure 3.1). Average stream gradients for Rainy and Fleetwood Creeks are 12.2% and 11.1% respectively.

The Rainy Creek drainage basin is located on a southern exposure of the Purcell Mountains, and is primarily forest covered except for the area disturbed by the mining/milling operations and logging operations. The basin rises from an elevation of approximately 2900 at the surface of the tailings impoundment, to 6040 feet at the top of Blue Mountain. The longest length of channel is about 4.9 miles for Rainy Creek, and about 3.1 miles for Fleetwood Creek. Average channel slopes are 5 to 15 percent, with sideslopes ranging from 5 to 45 percent. Rainy Creek enters the Kootenai River approximately 2 1/2 miles downstream of the tailings dam.

Mean annual precipitation at Libby is 19.4 inches, with 37 percent of it occurring in the months of November through January, and 18 percent falling in the months of May and June. The month having the highest average precipitation is January with 2.42 inches.

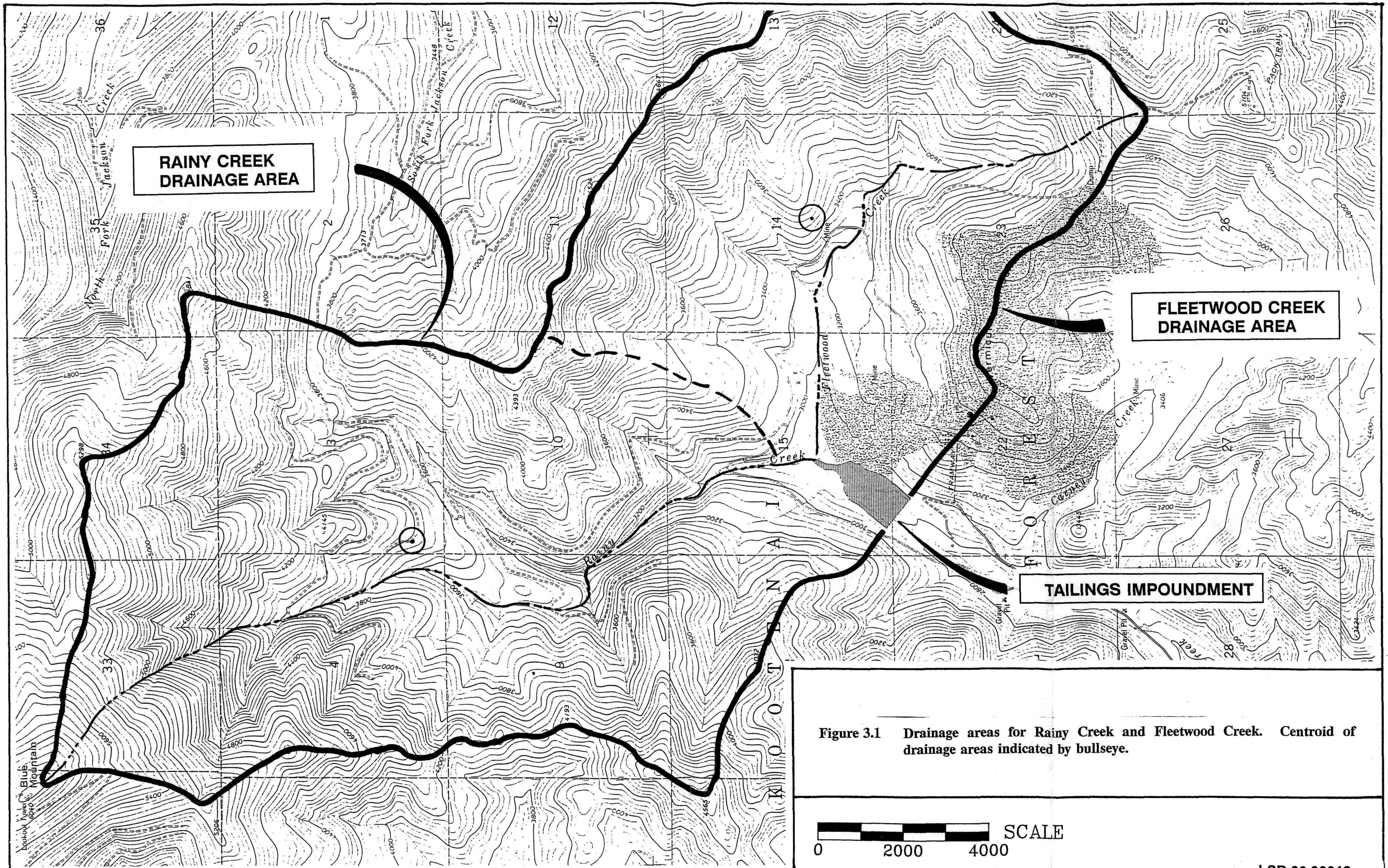


Figure 3.1 Drainage areas for Rainy Creek and Fleetwood Creek. Centroid of drainage areas indicated by bullseye.

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Temperature in Libby ranges from an average of 22.4° Fahrenheit (F) in January to an average of 67°F in July. Average annual precipitation at the site is estimated at 30 inches per year (USDA, 1977), and the temperature would be expected to average 3 to 5 degrees cooler than at Libby. Climatological data was obtained from the Libby 1 N.E. Ranger station.

Soils in the area have been assigned a Hydrologic Soil Classification of "B" by the Soil Conservation Service (SCS). The drainage basin is estimated to have >75% ground cover of mature forest in good condition, with moderate slopes. Antecedent moisture is considered to be average. A "Curve Number" of 60 is estimated for both the Rainy Creek drainage basin and the Fleetwood Creek drainage basin. As discussed in Section 3.2, Curve Numbers are used in the SCS hydrologic model to classify the drainage characteristics of different terrains. To assure a conservative runoff estimate, the curve number was selected slightly higher than normally recommended for forested lands to account for the impact from mining on areas of the Fleetwood Creek drainage and extensive clear cuts in Upper Rainy Creek. A summary of design conditions is shown in Table 3.1.

Table 3.1. Hydrologic parameters for Rainy Creek and Fleetwood Creek drainage areas impounded by the tailings dam.

WATERSHED NAME	AREA (sq. miles)	SCS CURVE NUMBER	AVE. SLOPE (%)	CHANNEL LENGTH (ft)	SOIL GROUP
Rainy Creek	5.9	60	12.2	25,870	B
Fleetwood Creek	3.5	60	11.1	16,370	B

3.2 DESIGN STORMS

Runoff from three design storms was used to evaluate flood routing through the tailings impoundment, specifically 1) a 10-year frequency, 24-hour precipitation event; 2) a 100-year frequency, 24-hour precipitation event; and, 3) a 6-hour probable maximum flood (PMF).

A spreadsheet program developed by Schafer and Associates was used to simulate the runoff from the 10 year and 100 year, 24 hour precipitation events. The model uses the calculation procedures outlined in the SCS National Engineering Handbook, Section 4, Hydrology (NEH-4). The SCS method finds a watershed flow hydrograph using the "Curve Number" method. A complete description of the background, methods and procedures is given in NEH-4 (U.S. Dept. of Agriculture, 1985). A brief description is provided below.

The SCS Curve Number Method was developed for areas having little rainfall data, particularly for storm duration and intensity. Runoff does not begin until after some period of "initial abstraction" (Ia) where infiltration, interception, and surface storage occur. The Ia is estimated to be 20 percent of the maximum potential runoff. Rainfall-runoff relations, based on SCS curve numbers, are then developed to estimate the runoff volume and timing from a precipitation event.

Curve numbers are selected based on land use, soil type, cover, hydrologic condition and antecedent moisture (see Section 3.1). Other necessary information includes average slope, drainage area and longest runoff length, and rainfall distributions as a SCS Type II convective thunderstorm event. Lag time, time of concentration, time to peak, etc. are calculated from the curve numbers. A series of elemental hydrographs, based on peak flows and the values of the dimensionless unit hydrograph (SCS), are developed for each duration, which in turn are summed to produce a total hydrograph. See Sections 3.2.1 and 3.2.2.

The PMF was calculated using the method outlined in the Department of Interior, Flood Hydrology Manual (U.S. Dept. of the Interior, 1989). The method is based on development of a "Synthetic Unit Hydrograph" which is used to estimate surface runoff from probable maximum precipitation. A brief description is given in section 3.2.3.

3.2.1 10-Year Event

A 10-year, 24-hour antecedent storm precipitation of 2.4 inches for Rainy Creek drainage basin was obtained from the National Oceanic and Atmospheric Administration (NOAA) Atlas (U.S. Dept. of Commerce, 1973). Using this precipitation value, and the boundary conditions outlined in Sections 3.1, a peak runoff for Rainy Creek (65 cfs) occurred 16.3 hours after the beginning of the storm. Peak runoff for Fleetwood Creek (45 cfs) occurred at 14.9 hours. Model results for the runoff of each drainage area are found in Appendix A. Key parameters for this model are summarized in Table 3.2. Figure 3.2 is a graphical representation of the surface water runoff and rainfall intensity for a 10-year, 24-hour event.

The total runoff hydrograph for the entire watershed area impounded by the tailings dam was obtained by summing the two individual hydrographs, resulting in a peak flow of about 107 cfs occurring at 15.5 hours after the beginning of the event. The total runoff for the affected drainage area is 74 acre-ft, with 46 acre-ft from Rainy Creek, and 28 acre-ft from Fleetwood Creek.

3.2.2 100-Year Event

A 100-year, 24-hour antecedent storm precipitation of 3.4 inches was obtained from the NOAA Atlas (U.S. Dept. of Commerce, 1973). Using this precipitation value, and the boundary conditions outlined in Sections 3.1, a peak runoff for Rainy Creek (262 cfs) occurred 15.2 hours after the beginning of the storm. Peak runoff for Fleetwood Creek (204 cfs) occurred at 14.4 hours as summarized in Table 3.3. Model results for the runoff of each drainage area are found in Appendix A. Figure 3.3 shows the surface water runoff and rainfall intensity for a 100-year, 24-hour event.

Table 3.2. **Surface water runoff for a 10-year, 24-hour precipitation event using SCS Type II rainfall distribution.**

10-YEAR, 24-HOUR STORM EVENT				
WATERSHED NAME	PRECIPITATION (inches)	RUNOFF (inches)	PEAK FLOW (cfs)	TIME OF PEAK (hrs.)
Rainy Creek	2.4	0.147	65	16.3
Fleetwood Creek	2.4	0.147	45	14.9
Combined Flows	2.4	0.147	107	15.5

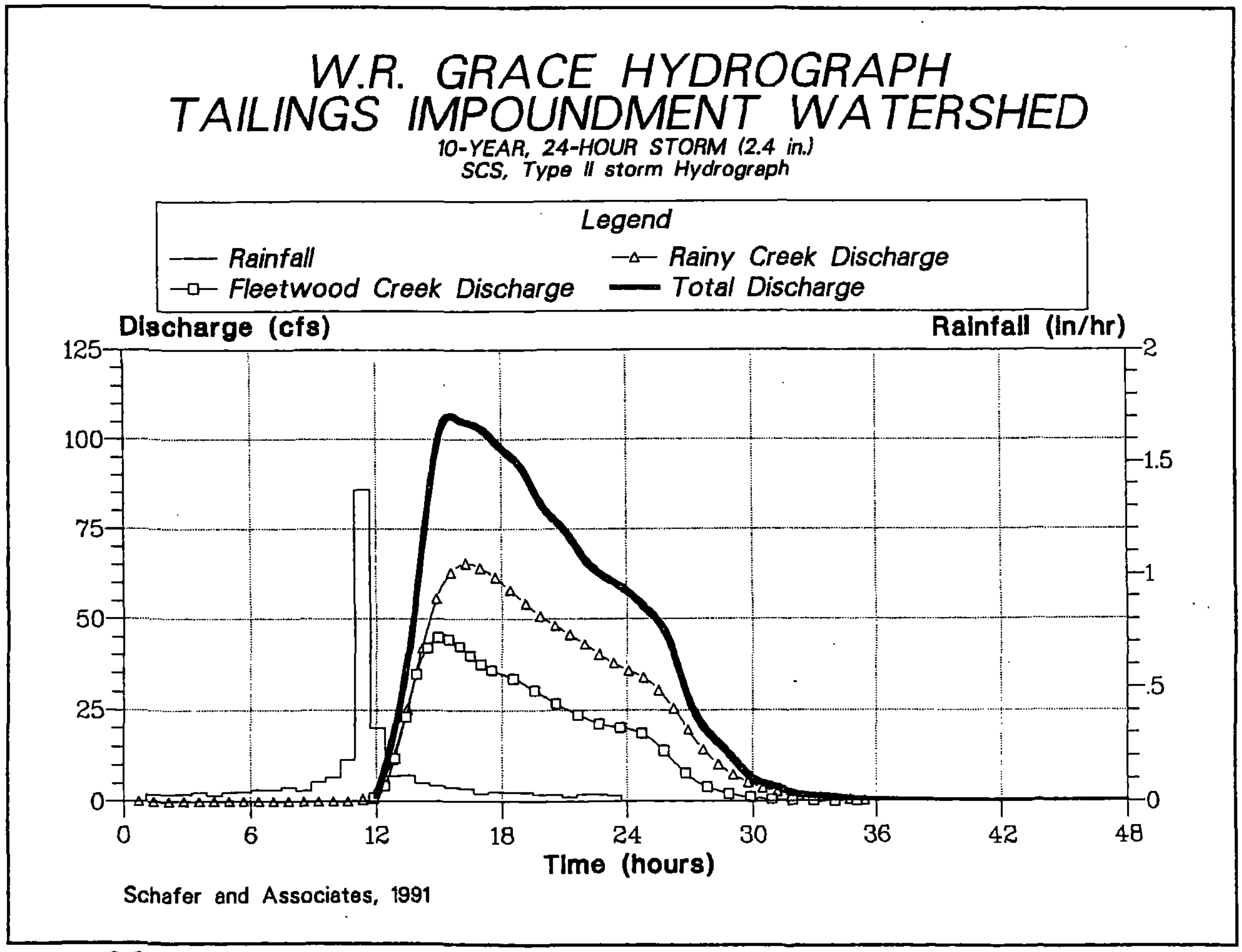


Figure 3.2 **Surface water runoff hydrographs and rainfall intensity for a 10-year, 24-hour storm (2.4 in.) in the Rainy Creek and Fleetwood Creek watersheds.**

Table 3.3. **Surface water runoff for a 100-year, 24-hour precipitation event using SCS Type II rainfall distribution.**

100-YEAR, 24-HOUR STORM EVENT				
WATERSHED NAME	PRECIPITATION (inches)	RUNOFF (inches)	PEAK FLOW (cfs)	TIME OF PEAK (hrs.)
Rainy Creek	3.4	0.489	262	15.2
Fleetwood Creek	3.4	0.489	204	14.4
Combined Flows	3.4	0.489	460	14.8

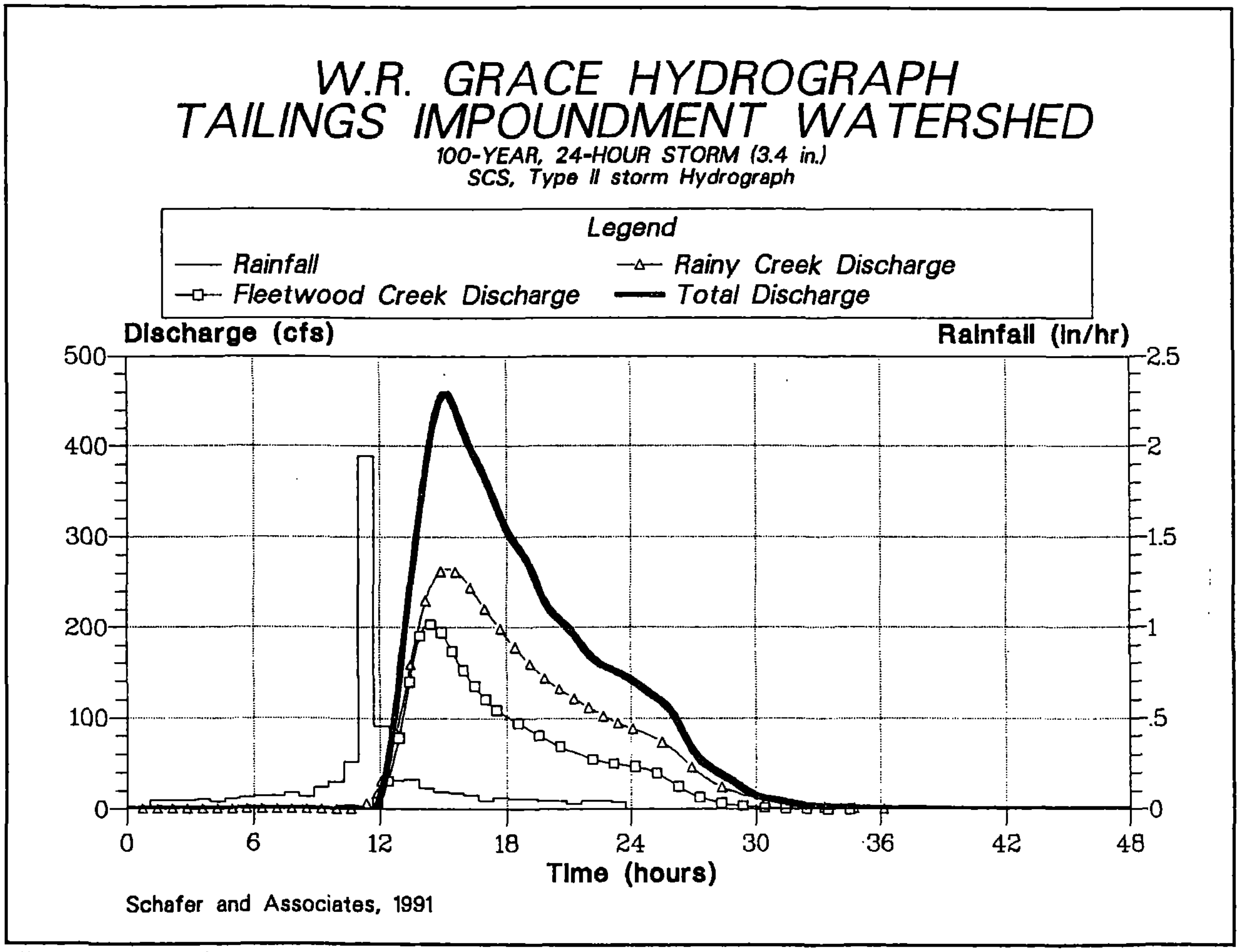


Figure 3.3 **Surface water runoff hydrographs and rainfall intensity for a 100-year, 24-hour storm (3.4 in.) in the Rainy Creek and Fleetwood Creek watersheds.**

The total runoff hydrograph for the entire watershed area impounded by the tailings dam was obtained by summing the two individual hydrographs, resulting in a peak flow of 460 cfs occurring at 14.8 hours after the beginning of the event (Fig. 3.3). The total runoff for the drainage area is 245 acre-ft, with 154 acre-ft from Rainy Creek, and 91 acre-ft from Fleetwood Creek.

3.2.3 Probable Maximum Flood

The probable maximum flood (PMF) is the flood expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in a region. Three scenarios are most often considered when estimating the PMF, specifically 1) general seasonal storms (October through June), 2) rain on snow (including snowmelt) and, 3) summer convective thunderstorms. Based on the Hydrometeorological Report No. 43 (HMR 43), "Probable Maximum Precipitation, Northwest States" (U.S. Weather Bureau, 1966), intense local summer thunderstorms of short duration are most likely to produce a PMF event in this region of the United States (east of the Cascade divide and west of the Rocky Mountains).

Using the method outlined in HMR 43 for summer thunderstorms in small drainage basins (<550 square miles), a PMF event is estimated to produce 10.7 inches of precipitation in 6 hours, distributed as shown by the hyetograph in Figure 3.4. Detailed calculations used to determine the PMF hyetograph are located in Appendix B.

Runoff from the PMF is calculated using the method outlined in the Bureau of Reclamation "Flood Hydrology Manual" (U.S. Dept. of the Interior, 1989). This method is similar to the SCS method described in Section 3.1, with the exception of the runoff determined by a synthetic unit hydrograph instead of summing a series of dimensionless unit hydrographs (SCS method). Input data requirements are similar, including drainage area, channel length, average slope, and ultimate infiltration (based on the SCS hydrologic soil group). As in the SCS method, lag time, duration, and incremental runoff are calculated from the input data. Input conditions are similar to those found in Section 3.1, with the exception of antecedent moisture conditions considered to be near or at saturation.

Important runoff parameters for this event are summarized in Table 3.5. The peak runoff for a PMF event in the Rainy Creek drainage area was calculated to be 7330 cfs, occurring 5.5 hours after the beginning of the storm. Peak runoff for Fleetwood Creek was calculated at 5884 cfs occurring at 4.5 hours after the beginning of the storm. Detailed calculations of the PMF runoff are located in Appendix B.

The total PMF runoff hydrograph for the entire watershed area impounded by the tailings dam was obtained by summing the two individual hydrographs (Rainy and Fleetwood Creeks), resulting in a peak flow of 11,676 cfs occurring at 5.0 hours after the beginning of the storm event (Figure 3.5). The total runoff for the drainage area is 4612 acre-ft, with 2895 acre-ft from Rainy Creek, and 1717 acre-ft from Fleetwood Creek.

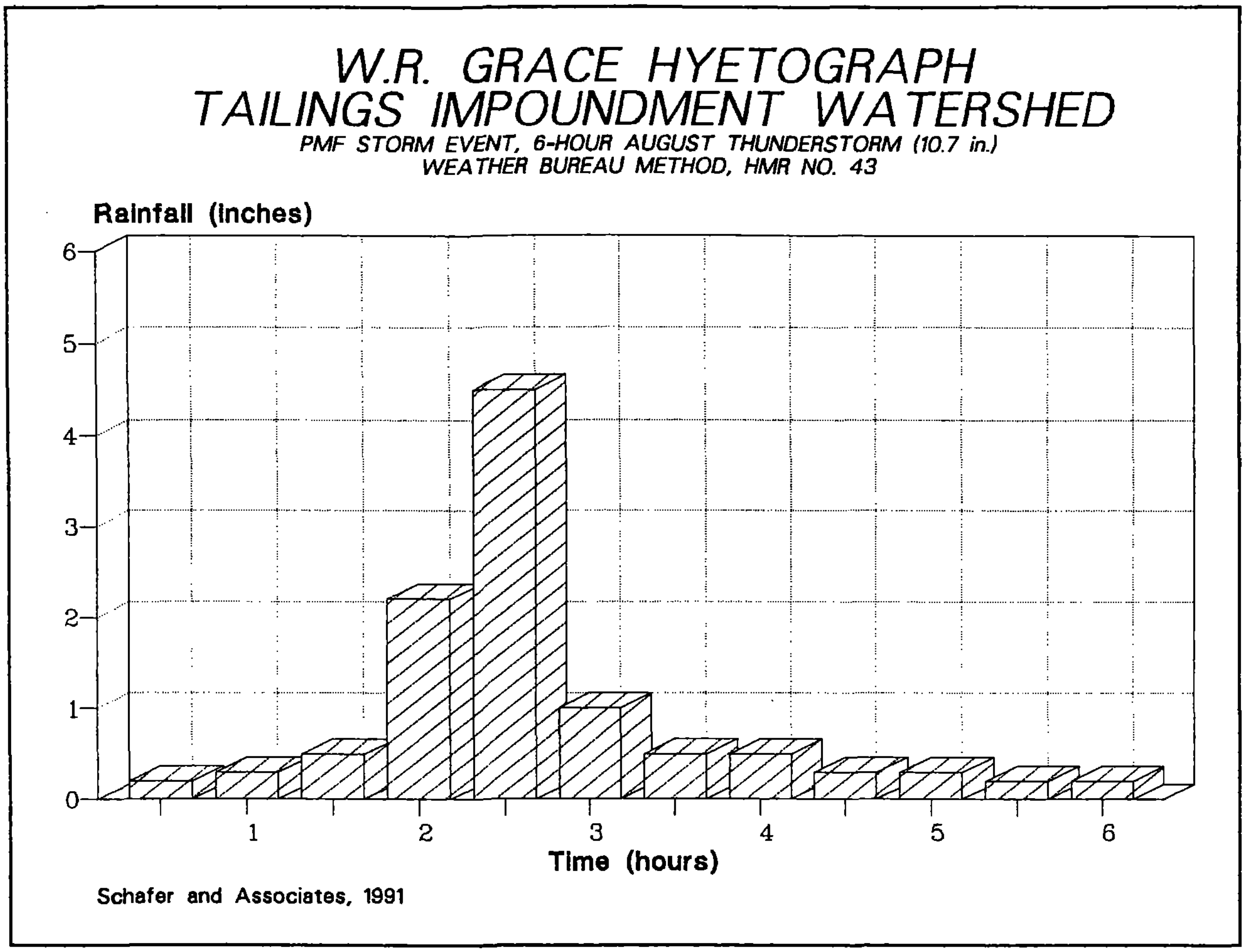


Figure 3.4 Storm hyetograph for a 6-hour PMF event (10.7 in.) in the Rainy Creek and Fleetwood Creek drainage basins.

Table 3.4. Surface water runoff for a 6-hour PMF event (10.7 in.) using the storm distribution hyetograph of Figure 3.4.

PROBABLE MAXIMUM FLOOD (6-HOUR)				
WATERSHED NAME	PRECIPITATION (inches)	RUNOFF (inches)	PEAK FLOW (cfs)	TIME OF PEAK (hrs.)
Rainy Creek	10.7	9.20	7330	5.5
Fleetwood	10.7	9.20	5884	4.5
Combined Flows	----	----	11676	5.0

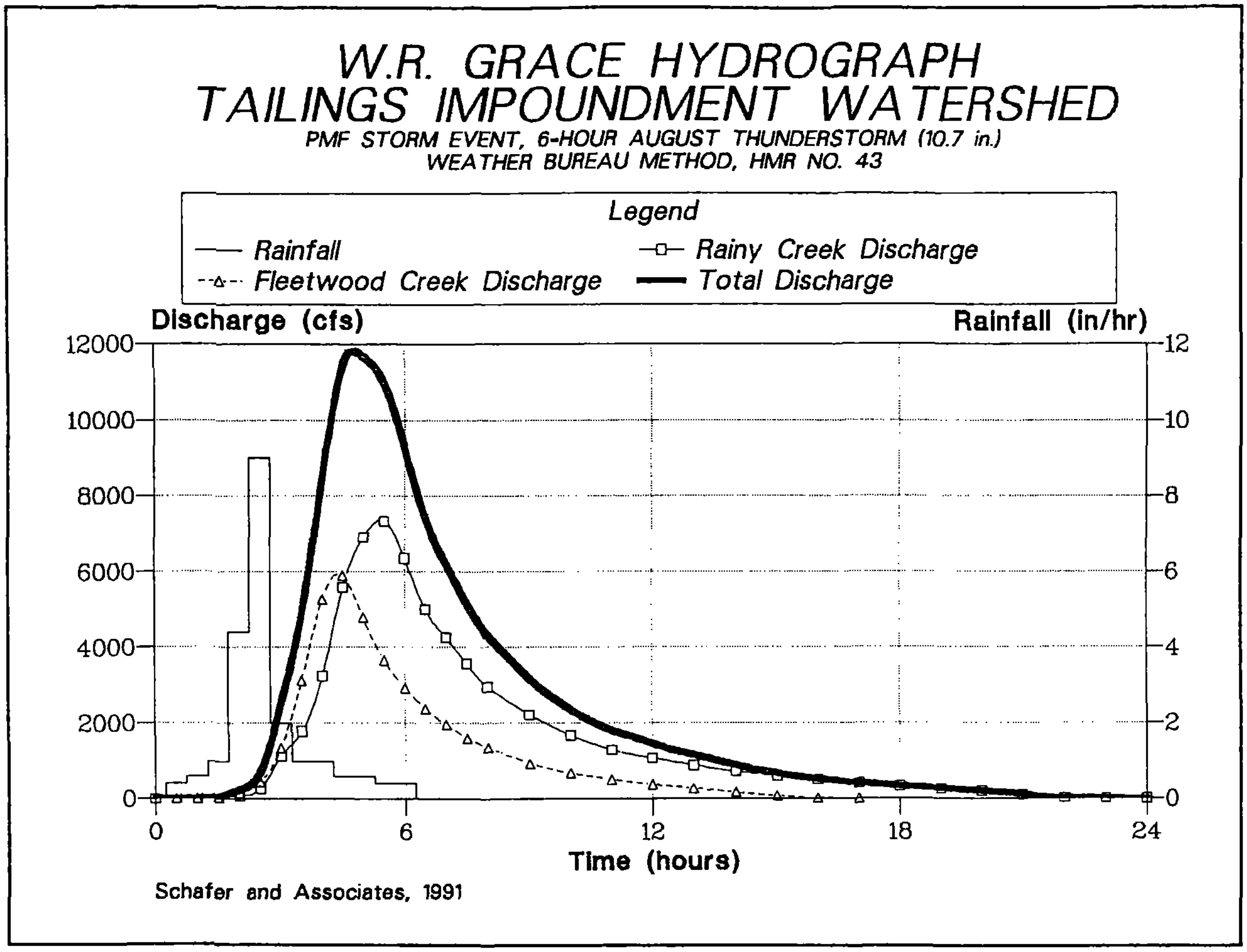


Figure 3.5 **Surface water runoff hydrographs for a 6-hour PMF event (10.7 in.) in the Rainy Creek and Fleetwood Creek watersheds.**

3.3 TAILINGS IMOUNDMENT CAPACITY

The top of dam elevation of the vermiculite tailings dam is 2926, with an existing emergency spillway crest elevation of 2920. The top of tailings elevations range from a low of 2895 just north of the decant tower, to a high of 2914 at the southeast corner of the impoundment. Average tailings elevation is estimated to be slightly over 2900.

Using the conic (volume) method to determine the reservoir storage capacity, it is estimated that the reservoir will have a surface area of 68.7 acres and a storage volume of 871 acre-feet measured to the crest of the (existing) emergency spillway. Approximately 431 acre-feet of storage is available between the existing emergency spillway crest and the dam crest, making the total storage capacity (top of dam) 1302 acre feet. A tabulation of impoundment capacities as a function of elevation is given in Table 3.5.

Table 3.5. Storage capacity of the tailings impoundment/reservoir.

ELEVATION (ft.)	AREA (acres)	INCREMENTAL VOLUME (acre-ft.)	CUMULATIVE VOLUME (acre-ft.)
2895[1]	-	-	-
2900	10.4	26.0	26.0
2905	21.0	78.5	104.5
2910	48.7	174.3	278.8
2915	59.7	271.0	549.8
2920[2]	68.7	321.0	870.8
2926[3]	74.9	430.8	1301.6

1 Lower limit of impoundment.
2 Emergency spillway crest elevation.
3 Top of dam elevation.

During the closure work on the impoundment, it is proposed that the existing emergency spillway will be removed, and a new emergency spillway constructed on the west side of the dam. The emergency spillway will work in conjunction with a proposed primary outlet/control structure to route flows through the reservoir. See Section 5.0 for details of the preferred alternative.

3.4 DAM SAFETY REQUIREMENTS

The Rainy Creek Basin Zonolite Tailings Dam, MT-1470 has been rated as large in size and as having a high downstream hazard potential (Category 1), as determined by an inspection and report completed by Morrison-Maierle in 1981 (Foster, 1981). The inspection was conducted in accordance with U.S. Army Corp of Engineers Guidelines for Safety Inspection of Dams, and was completed for the State of Montana Department of Natural Resources and Conservation, under Public Law 92-367. The classification is based on a dam height of 135 feet, and storage capacity of 2120 acre-feet at the spillway crest.

Under State of Montana regulations for Dam Safety, Rule 36.14.206 (State of Montana, 1989):

(1), "*.....hazard determination shall be based on the consequences of dam failure--not the condition, probability, or risk of failure. A dam must be classified high-hazard if the impoundment capacity is 50 acre-feet or larger and it is determined that a loss of human life is likely to occur within the breach flooded area as a result of failure of the dam.*"

(3) " ..... *the effects of flood inundation...... will continue downstream until the flood stage is equal to that of the 100 year floodplain.*", and

(5) "*Loss of life is assumed to occur if the following structures are present: .... other paved highways....*".

Under Rule 36.14.502:

(1) "*Spillways (principal and emergency) for high-hazard dams must safely pass the flood calculated from the inflow design flood. The minimum inflow design flood is expressed as a fraction of the probable maximum flood or as otherwise indicated in Table A*" (See Table 3.6),

(2) "...... *The minimum inflow design flood shall be the 100-year, 24-hour flood*",

(3) "......... *routing of the inflow design flood through the reservoir shall assume storage contents to be at the emergency crest elevation prior to routing*",

(4) "....*breach area ..... is designated as Category A if .... major repair or alteration of the emergency spillway is to be performed, where the downstream hazard contains more than 20 residences and the failure flood wave is less than 4 hours from the dam to the first residence*",

(5) "*.....breach area ...... is designated as Category B if the dam is an existing dam not meeting the criteria for a Category A dam*".

Table 3.6. Emergency spillway inflow design flood(s) from Table A of the Montana Dam Safety regulations, Rule 36.14.502.

CAPACITY TO THE EMERGENCY CREST/HEIGHT TO DAM CREST	BREACH AREA CATEGORY A	BREACH AREA CATEGORY B
Dams less than 100 acre-feet and less than 20 feet in height	2Q	Q
Dams less than 500 acre-feet and less than 35 feet in height	.2 PMF	.1 PMF
Dams less than 1000 acre-feet and less than 50 feet in height	.3 PMF	.15 PMF
Dams less than 12,500 acre-feet and less than 50 feet in height	.5 PMF	.5 PMF
Dams less than 50,000 acre-feet and less than 100 feet in height	.75 PMF	.75 PMF
Dams 50,000 acre-feet or greater and 100 feet or greater in height	1.0 PMF	1.0 PMF

With the top of tailings elevation of 2900+, the height to the crest of the dam (from the tailings surface) is less than 50 feet, and the capacity of the reservoir to the (existing) emergency spillway crest is less than 1000 acre-feet. The work will be considered to be a major alteration to an existing dam.

Based on the above criteria, the tailings dam is considered to be high-hazard, making it applicable to all other criteria for high-hazard dams. The breach area below the dam is unknown, therefore it will be considered as Category A. There are no residences between the dam and the Kootenai River, however, a paved highway does exist. The impact to the Kootenai River is unknown, but is not expected to exceed the 100 year floodplain at the closest residence downstream. The condition of the Kootenai River at the time of dam breach will be unknown. Based on these guidelines and criteria, the required design flow is 0.30 PMF, or 3504 cfs.

The flood routing volume proposed by W.R. Grace is 0.5 PMF, which calculates to a design value of 5838 cfs (0.5 x 11,676 = 5838). This 0.5 PMF value will be used during flood routing analyses.

3.5 PROPOSED DESIGN FLOWS

W.R. Grace proposes to use the flows summarized in Table 3.7 for flood routing through the vermiculite tailings impoundment. Boundary conditions and assumptions follow:

- A 2.4 inch, 24 hour design storm to simulate a 10-year return storm; and 3.4 inch, 24 hour design storm to simulate a 100-year return storm. Both storms are distributed as a SCS Type II convective thunderstorms;

- A 10.7 inch, 6 hour design storm to simulate a probable maximum flood (PMF) event, distributed as a convective thunderstorm according to U.S. Weather Bureau guidelines;

- Soils within the drainage classify as SCS type "B" soil group. The soils contain average in-situ antecedent moisture for the 10 year and 100 year return storms. Soils are considered to be near saturation, with 0.25 inch per hour infiltration for PMF event;

- The drainage basins are dense forest in good condition, with >75% ground cover;

- Curve numbers of 60 are used for both Rainy Creek and Fleetwood Creek drainage basins.

- The tailings dam is classified as a high-hazard dam according to Montana Dam Safety, and U.S. Army Corp of Engineers regulations;

LSB 66 00028

- The required inflow design is 0.30 PMF, based on less than 50 foot dam height (from surface of tailings), less than 1000 acre-feet storage at emergency spillway crest, and a Category A breach area (State of Montana, 1989);

- 0.5 PMF will be used for flood routing analyses and design;

- The existing tailings impoundment decant tower and emergency spillway, and the Rainy Creek diversion and pipeline will be removed during closure;

Table 3.7. Design flood volumes proposed for flood routing alternatives analysis and conceptual design.

WATERSHED NAME	10-YEAR, 24-HOUR (cfs)	100-YEAR, 24-HOUR (cfs)	0.5 PMF 6-HOUR (cfs)
Rainy Creek	65	262	3665
Fleetwood Creek	45	203	2942
Combined Flows	107	460	5838

4.0 FLOOD ROUTING

4.1 OVERVIEW OF ALTERNATIVES

The project calls for engineering analysis of available alternatives for routing floods through the area affected by the vermiculite tailings impoundment. Concerns that will be addressed by the analysis include safety, potential for water contamination especially from asbestiform fibers, long-term stability of the impoundment including an analysis of tailings dam saturation and seismic events, sedimentation, and others concerns.

Three basic options for flood routing have been considered: Alternate I - diverting all flows, including storms producing PMF events, around the impoundment and dam, Alternate II - routing flows through the impoundment and discharging through an outlet channel constructed in or near the dam and Alternate III - a partial diversion of "normal" stream flows and routing of events exceeding diversion design flows into the impoundment. Flood routings were modeled using a computer program entitled "Hydrograph Develop Program", developed by the SCS in 1990. Routing models were completed by Lew Burton and Ed Juvan, retired SCS engineers.

Within each of the general alternates are several design variations which have been considered in varying degrees of detail. Table 4.1 provides a summary of the pertinent features of each option considered. A discussion and evaluation of the alternatives follows in Sections 4.2 through 4.4. A description of design details for the preferred alternative is given in Section 5.0.

In the following investigations, each main alternative will begin with a discussion of general parameters, followed by specific routing alternatives, and finally a summary of advantages and disadvantages. Maps, sections, and other design drawings will be provided as necessary. The project area has been set up as a grid, with the north-south (horizontal) axis designated by letters (A - L), and the east-west (vertical) axis designated by numbers (1 - 9). This should provide for a more efficient method of locating sections or more detailed drawings. The base grid system is delineated on Plate 1.

Table 4.1 Summary of alternatives considered for flood routing.

Alternative	Essential Design Features
Full Diversion **Alternate Ia:** Partial Isolation of Tailings **Alternate Ib:** Total Isolation of Tailings **Alternate Ic:** West Side Diversion Channel **Alternate Id:** East Side Diversion Channel **Alternate Ie:** Pipeline	• Diversion dam(s) upstream of tailings dam to intercept streams • Flood routing in large channels around dam • Large drop chutes for return of stream flow to Rainy Creek below dam
Channel Reconstruction In Tailings **Alternate IIa:** Water Level at 2904' **Alternate IIb:** Water Level at 2910' **Alternate IIc:** East Abutment Outlet **Alternate IId:** West Abutment Outlet **Alternate IIe:** Outlet Over Dam Face	• Streams enter impoundment and collect in a pond at the upper end with water level kept away from dam for improved stability • Unused tailings impoundment capacity used for storm surge up to 0.5 PMF • Lined channel (for erosion control) delivers water to outlet structure at the dam • Box culvert outlet control structure reduces stream discharge from impoundment during major storm events • Optional emergency spillway for storms in excess of 0.5 PMF • Armored channel/drop structures return stream flow to Rainy Creek below the dam
Partial Diversion **Alternate IIIa:** 100-Year Storm Diversion **Alternate IIIb:** 10-Year Storm Diversion	• Diversion dam(s) upstream of tailings dam intercepts Rainy and Fleetwood Creeks • Outlet control structure reduces stream discharge from diversion dams to a design maximum which is routed around the tailings • Drop chutes similar to Alternate I but smaller return diverted stream flow to Rainy Creek below tailings dam • Runoff in excess of design maximum overflows to the tailings impoundment • Secondary outlet and discharge channel similar to that of Alternate II

4.2 FULL DIVERSION ALTERNATIVES

4.2.1 Description of Design Concepts

<u>Common Diversion Dam (Alternate Ia):</u> Diversion of Rainy and Fleetwood Creeks around the impoundment is one possible method of flood routing following closure. Full diversion will entail intercepting, diverting both creeks around the impoundment, and ultimately returning them to Rainy Creek downstream of the dam.

Construction of a diversion dam across the upper end of the existing impoundment would be required at a location where flows from Rainy Creek and Fleetwood Creek join. The flows would then be diverted around the tailings impoundment through an open channel or pipe constructed adjacent to the impoundment. Once past the dam, a concrete drop chute or other means of elevation reduction would return the diverted flows to Rainy Creek. Plate 2 is a conceptual plan view of this alternate.

A full diversion dam, capable of diverting a 0.5 PMF event while retaining long-term structural integrity, will be very difficult to construct because of the tailings in the impoundment and east abutment. Tailings will not provide a competent foundation for the dam base or abutment, hence significant excavation of the tailings would be required (see Plate 3). Conventional construction methods and equipment often fail when working in tailings, making the project costly and with questionable results.

<u>Separate Diversion Dams (Alternate Ib):</u> An alternative would be to construct a diversion dam at the extreme upper end of the impoundment, beyond the extent of the tailings. A separate diversion dam would be constructed for Fleetwood Creek upstream of the coarse tailings dump. Flows from Fleetwood Creek would be delivered to the Rainy Creek diversion by a constructed channel (Plate 4-A). Both flows would then enter a main diversion channel and be routed around the impoundment as above (Plate 4-B)

<u>West Side Channel (Alternate Ic):</u> Should full diversion be selected, the best method for carrying the diverted flows around the tailings impoundment would be an open channel constructed on the west side of the impoundment. The channel would be constructed in natural material (off the tailings), and connected to a concrete drop chute/plunge pool below the tailings dam. Flows would be diverted into the constructed channel at the diversion dam, carried around the tailings dam and impoundment, and returned to Rainy Creek downstream of the dam. Refer to Plates 2, 4-A, and 4-B.

A conceptual design was completed for a 0.5 PMF channel on the west side of the tailings using a beginning channel elevation of 2900.0, and a gradient of 0.005 ft/ft (0.5%). The structure would be a rock-lined, trapezoidal open channel with 20 ft wide (flat) bottom and 2:1 sideslopes. With a design flow of 0.5 PMF (5838 cfs) and applying Manning's Equation:

$$Q = A\frac{1.486}{n}R^{2/3}S^{1/2}$$

in which:

Q = volume of flow, cfs
A = cross-sectional area of flow in ft^2
S = slope, ft/ft
R = hydraulic radius, ft
n = coefficient of roughness (0.04 for rock lined channels)

a peak flow depth of about 12 feet is calculated with a velocity of approximately 11 feet per second. With the beginning channel elevation of 2900 and 0.005 ft/ft gradient, the bottom elevation of the channel opposite the dam will be about 2888. Recommended maximum cut slopes are 2:1, with spaced 10 ft safety benches where possible. The channel would be armored with a minimum of 24 inches of D_{50} = 18 inch-rock lining to handle the velocities associated with peak flows corresponding to the predicted peak water level. Plate 5 shows a typical cross-section of the west side diversion channel (relative location shown on Plate 2).

East Side Channel (Alternate Id): An alternate full diversion channel would be to construct an open channel on the east side of the impoundment. The channel would be similar to the west side with a concrete drop chute/plunge pool. Flows would be diverted into the channel at the diversion dam, carried around the impoundment, and returned to Rainy Creek downstream of the tailings dam.

This alternate is not practical due to the proximity of the coarse tailings dump, and presence of shallow bedrock and steep slopes. The beginning section of the channel would be located entirely within the coarse tailings dump which is unconsolidated and geotechnically unstable. Significant design and engineering would be necessary to construct a channel in this material. Further, lining would be required to prevent rapid infiltration and increased foundation instability. Excavation to natural material would be virtually impossible.

On the lower sections of the channel, the depth to bedrock is generally less than 10 feet (Lewis, 1971) and portions of the drainage sideslopes are very steep. These restrictions, coupled with the required channel size for 0.5 PMF, would require that the channel be constructed partially within the fine tailings (see Plate 6). An alternative would be to construct the channel entirely in bedrock (see Plate 7), requiring extensive drilling and blasting. Either channel location has drawbacks.

Pipeline (Alternate Ie): A pipeline, or other closed conduit, was explored as an alternate for carrying full diversion flows around the tailings impoundment. As with the open channels, the entire flow from both Rainy and Fleetwood Creeks would be diverted into the pipeline which would carry this flow around the impoundment and return it to Rainy Creek downstream of the tailings dam. The pipeline would most likely be located on the west abutment, and would eliminate the need for a drop structure.

The primary advantage to a pipeline is the elimination of water loss through infiltration, and associated tailings dam saturation problems. Another advantage is the reduction in public accessibility, with the exception of the pipe entrance.

Disadvantages include size, geotechnical stability, maintenance, and cost. A pipeline greater than 20 ft (diameter) is required to carry 5838 cfs, the exact size depending on shape and material type. To properly install a pipe of this size requires extensive excavation, and specialized construction methods and equipment. Pre-stressed concrete pipe would be the best choice, but with considerable cost. Even with pre-stressed concrete, geotechnical stability may remain a problem, due primarily to the geology and topographic relief of the area.

A safety concern is the entrance into the pipeline, and the closed system preventing quick escape. Installation of a grate, or other barrier would prevent this, but would greatly increase maintenance and the possibility of plugging with subsequent system failure during major events.

4.2.2 Evaluation of the Full Diversion Alternatives

<u>Safety:</u> Safety and long-term integrity of any system are directly related, and should be the primary considerations when selecting a flood routing system. The full diversion alternate <u>increases</u> the potential for failure, and <u>decreases</u> the safety of the system. The drop chute and plunge pool, constructed of reinforced concrete, would be difficult to build on steep slopes such as these. Stability of the structure in a massive flood condition would be problematic.

The channels carrying the diverted flows would be very large, and inherently less stable than smaller channels, particularly when constructed into the side of a hill as they would be in this case. From a hydrologic and geotechnical standpoint, any channel, natural or constructed, located above the low point in a drainage is generally not considered to provide good long-term service, particularly when considering flows of this magnitude.

For the east side diversion channel, the combination of construction difficulties and doubtful foundation/geotechnical factors make this alternative a poor choice for a long-term diversion channel. For both east and west side channels, construction of the drop chute will be costly, and plugging during high flows a primary concern.

The drop chute below the tailings dam would be a large, concrete structure to handle the volume and velocity of the peak flows. Construction on the steep terrain of the west abutment area will be very expensive, and long-term geotechnical stability may be difficult to obtain. Other safety considerations include public accessibility to the large, fast moving flows in the channels and drop chute, and the difficulty in "escaping" from such.

The diversion dams are designed to only collect water prior to routing around the impoundment and would have little useful storage capacity. Should the diversion channels become plugged, or the system fail for some other reason, the flood flows would quickly breach the diversion dams and enter the impoundment. The breach could be rapid, in turn causing a large surge of water to strike the tailing dam. If the tailings dam did not fail from impact, the impoundment would begin to fill and could cut a new channel from the tailings impoundment into the diversion channel, or in an improbable event, could block the diversion channel with debris so badly that overtopping of the impoundment might occur. Either event would bring the potential for extensive uncontrolled erosion of the tailings material. Overtopping the dam could cause catastrophic failure of the dam unless additional precautions are taken. Dams in a series are not considered to be good engineering practice.

Full diversion of a 0.5 PMF event (producing 5838 cfs) requires a complex system of very large diversion dams, channels, and drop chute to route the <u>entire</u> peak flow of a storm of this magnitude around the impoundment, and return it to Rainy Creek downstream of the tailings dam. This alternate ignores the potential for flood control in the unused storage capacity of the impoundment. By allowing the reservoir to surge and temporarily store the peak flood flows, outflow peaks can be reduced to roughly 15 percent of the peak inflow (5838 cfs) and still contain a 0.5 PMF event.

<u>Water Quality Impacts:</u> While water contamination, particularly from tremolite fibers <u>may</u> be reduced by diversion, it will <u>not</u> be eliminated. Constructing a diversion dam to collect both flows simultaneously will include a section of the tailings impoundment. In addition, Fleetwood Creek will be flowing through the coarse tailings.

Asbestiform fiber contamination from the tailings impoundment and coarse tailings dump could be eliminated by the second diversion alternative shown in Plate 4-A and 4-B. This alternative would prevent streamflows from contacting the tailings, however, these fibers would continue to enter Fleetwood Creek from the natural vermiculite intrusive from which Fleetwood Creek originates. Further, Carney Creek, which enters Rainy Creek downstream of the impoundment, will continue to contribute tremolite fibers to Rainy Creek, regardless of the routing alternative selected.

<u>Environmental Impacts:</u> Environmental disturbance would be significant for a full diversion flood routing system, primarily from the massive excavations required to construct the diversion channels and drop chute. Environmental disturbance would be less for the east side channel than the west channel, but still significant. Channel lining with an impermeable material is recommended to prevent the complete loss of the smaller summer flows, and reduce potential for dam saturation. In order to construct an engineered channel that would have a reasonable longevity and acceptable maintenance, a large portion of either abutment would be removed, which creates an additional problem, namely, where to spoil the waste.

Additional concerns include relocation of the Forest Service access road at several locations, and the continued downstream flooding and erosion from the full 0.5 PMF flows.

<u>Tailings/Dam Saturation:</u> Saturation of the tailings dam and subsequent seepage and instability in the event of toe drain failure has been identified as a major regulatory concern. This subject has been addressed in detail by a study completed by Harding Lawson Associates (HLA) of San Francisco, California (Vahdani, 1992).

HLA completed a drilling program in the tailings and in the dam foundation materials as part of a study to assess the stability of the dam and impounded tailings during static and seismic loading conditions. The study concludes that the dam is currently safe under seismic load, even with the water at the face of the dam, and will not fail. The study encountered two types of tailings materials which appear to be interbedded and sloping away from the dam face. Elastic silts comprise about 60 percent of the tailings while loose, poorly graded sands and silty sands comprise about 40 percent. The elastic silts were not expected to liquify in a seismic event; however the sands could liquify if they remain saturated. If a section of the dam were to be removed the tailings could be expected to fail, but would maintain a 4:1 angle of repose. HLA judges the potential for material run-off in the event of a failure to be very low on the basis of its findings.

The drilling also indicated that the tailings consolidated with depth and gained significant strength. If the tailings are left without standing surface water, up to 5 feet of surface subsidence is projected in areas of deeper tailings as excess pore water pressure is relieved. HLA sees the major threat to dam stability to be the eventual failure of the toe drain piping. It will then be possible for the phreatic surface to increase in the dam and possibly begin seeping from the dam face. Should this occur there will be the likelihood of erosion of the toe and eventual weakening of the dam. Installation of additional piezometers is recommended to provide better monitoring and a conceptual design for a permanent drain structure to be retrofitted as required is proposed. HLA has indicated that based on the probable hydraulic conductivity of the tailings material, it may be possible to reduce the phreatic surface in the dam permanently by maintaining the pond surface approximately 500 feet upstream from the crest of the dam.

Diverting flows around the tailings impoundment will <u>not</u> eliminate saturation of a portion of the tailings dam adjacent to the channel, unless an impermeable liner is installed. The material covering the bedrock on the abutments is glacial outwash and till with moderate to very high permeability (Lewis, 1971). Significant loss of water through infiltration would be expected. The area of influence from the lost water is unknown but is likely to impact a portion of the tailings dam.

Infiltration could be eliminated by lining the channel with an impervious liner material, possibly HDPE or clay. Depending on the life of the selected material, infiltration would be significantly reduced or eliminated, at least through the life of the liner. Channel lining is an option with each alternative, hence no advantage or disadvantage to a particular alternative.

<u>Sedimentation:</u> Reduction of downstream sedimentation associated with the tailings would be expected with a full diversion, particularly if the second (full diversion)

<u>Sedimentation:</u> Reduction of downstream sedimentation associated with the tailings would be expected with a full diversion, particularly if the second (full diversion) option were exercised. The surface of the impoundment is currently devoid of vegetation and subject to potential erosion in major storm events despite its relatively low angle of repose because of the small particle size of the fine tailings. Over the next three to five years, it is anticipated that vegetation will become firmly established on the both the fine and coarse tailings and the potential for erosion will be greatly decreased. Sediment contribution from the tailings should become relatively insignificant.

Disadvantages associated with diversion include the loss of settling and natural filtration associated with some of the other options which provide a wetland in the upper portion of the tailings impoundment. While the impact of sedimentation from tailings materials may be lessened, there is a great potential for increased sedimentation from other sources associated with the massive excavations which would be required for the channels, drop chute, and other diversion structures.

In summary the full diversion alternates greatly reduce safety, increase the possibility of system failure, increase environmental disturbance, and increase construction and maintenance costs. Concerns over geotechnical stability, asbestiform fibers, and tailings dam saturation are not eliminated.

Advantages (Table 4.2) and disadvantages (Table 4.3) of the full diversion alternates are summarized below:

Table 4.2. Advantages associated with a full diversion flood routing system.

ALTERNATE	FULL DIVERSION - ADVANTAGES
All Alternates	• <u>Possible</u> reduction in downstream tremolite fiber concentration in surface water; • Probable reduction in short-term sedimentation from the tailings impoundment.
Common Diversion Dam (Alternate Ia)	• Provides the least complex design for intercepting flows from both Rainy Creek and Fleetwood Creek.
Separate Diversion Dams (Alternate Ib)	• Intercepts water from both Rainy Creek and Fleetwood Creek before contact with any portion of the tailings impoundment area.
West Channel (Alternate Ic)	• Best overall alternate of full diversion channels; • Most stable geotechnically of the full diversion alternates.

ALTERNATE	FULL DIVERSION - ADVANTAGES
East channel (Alternate Id)	• Less environmental disturbance than west channel; • Bedrock channel reduces infiltration and subsequent potential for saturation of tailings dam.
Pipeline (Alternate Ie)	• Eliminates infiltration and subsequent saturation of tailings dam; • Least (long-term) environmental disturbance of diversion alternates; • Least public accessibility to flood flows, excluding inlet; Eliminates need for separate drop chute.

Table 4.3. Disadvantages associated with full diversion flood routing system.

ALTERNATE	FULL DIVERSION - DISADVANTAGES
All Alternates	• Does not use the reservoir capacity to temporarily store peak flows resulting in higher peak flows downstream in Rainy Creek; • Will not eliminate tremolite fiber contamination of downstream surface water; • Construction of diversion dams in tailings is very difficult, and the long-term stability of such dams is questionable; • Significant environmental disturbance to construct channels/pipeline to carry diverted flows around the tailings impoundment. Massive cut slopes would be required; • Diversion dam(s), channels, and other structures will be required to handle 0.5 PMF flows, making them large and very costly; • Dam safety is inferior. Dams in series are more prone to catastrophic failure; • No backup flood routing system; • The diversion channels will not eliminate the possibility of tailings dam saturation and resultant stability concerns, unless impermeable lining is installed; • Does not take advantage of the wetland within the tailings facility for settling and natural surface water filtration; • Increased maintenance; • Tailings will be dry, thereby increasing the possibility of blowing dust and raising air quality risks; • Limited opportunity for wetland habitat construction.

ALTERNATE	FULL DIVERSION - DISADVANTAGES
Common Diversion Dam (Alternate Ia)	• Does not achieve complete isolation of streamflows from tailings materials.
Separate Diversion Dams (Alternate Ib)	• More complex design; • Greater environmental disturbance resulting from construction.
West channel (Alternate Ic)	• Significant environmental impact from the massive excavations required to properly construct a long-term channel; • Channels are prone to plugging with debris, particularly during flood events, resulting in greatly increased risk of channel/system failure and associated safety risks; • Channel would require lining to prevent infiltration into underlying material, particularly during low flows; • Major relocation of the Forest Service access road would be required.
East channel (Alternate Id)	• Upper reach of channel in geotechnically unstable coarse tailings material; • Lower portion partially within fine tailings, or would require drilling and blasting of bedrock to construct channel; • Channel would be prone to plugging with associated safety risks; • Construction difficulties; • Channel lining would be required in coarse tailings section to prevent water loss and foundation problems; • Significant environmental impact, although less than west channel.
Pipeline (Alternate Ie)	• Very large (>20 ft diameter) pipe required to carry the full 0.5 PMF design flows; • Very expensive construction and material costs; • Geotechnical stability questionable; • Considerable maintenance required; • Prone to plugging, and once plugged, very difficult to clean; • Safety concern (no escape) from a closed system.

4.3 CHANNEL RECONSTRUCTION THROUGH THE IMPOUNDMENT

Initial studies of this concept were approached from the standpoint that a 0.5 PMF event could be safely routed through the impoundment and discharged through the dam into a channel or drop structure constructed to withstand such a massive flood event. Large, armored channels similar to those required for a full diversion were the result. These concepts suffered from many of the same stability problems that were cited for the full diversion alternatives. A study of the flood surges and the damping effect caused by the unfilled volume of the tailings impoundment suggested that the most useful feature of this concept is the potential for storing much of the runoff from events comparable to a PMF and releasing it downstream at a much reduced and more manageable volume. Our investigation centered on designs which would take advantage of this as it provided the safest method of passing a flood event equal to or exceeding a 0.5 PMF event, while adequately addressing the majority of the engineering, environmental and geotechnical concerns.

A general concept employed in these alternates is to hold water away from the dam during all but very large runoff events. This principal of design results from the work of Harding Lawson Associates on geotechnical stability of the dam. The study showed that although the dam would not fail with water at the face even during an earthquake, additional stability and a reduced risk of foundation saturation could be obtained by keeping water back some distance thereby lowering the phreatic surface at the dam. We considered two concepts for providing this increased level of stability and several options for passing water through the dam face. These alternatives are described in Section 4.3.1 below.

4.3.1 Description of Conceptual Designs

Water Level at 2904' (Alternate IIa): This alternate would allow inflows from Rainy and Fleetwood Creeks to enter the impoundment unimpeded. Once in the reservoir, the flows would be temporarily stored, or passed directly through the impoundment with a constructed channel, depending on the volume received. This alternate provides for a water elevation in the impoundment of 2904 feet which is the minimum practical elevation that can currently be obtained through control at the decant tower. Tailings materials have accumulated to this level at the decant tower.

Discharge from the impoundment would be controlled at the tailings dam by a control structure, preferably a single concrete box culvert. The control structure would limit outflows to a maximum design flow (about 15 percent of 0.5 PMF). At this design rate the impoundment can receive a 0.5 PMF event without overtopping the dam.

An extensive study of outlet control structures was made before selecting the box culvert design. The control structure must necessarily have a small cross-sectional area if it is to reduce the volume of discharge and fully utilize the impoundment storage capacity. More natural control structures such as open channels were considered but these could only be utilized by sacrificing a large portion of the impoundment's potential storage capacity. Pipelines were also considered as an inexpensive alternative but these presented safety hazards and were judged to be more subject to failure in long-term service.

Outflows from the control structure would be returned to Rainy Creek by an engineered channel armored with a rock rip-rap lining, integrating a series of reinforced concrete drop structures. The channel would be considerably smaller than a full diversion channel, and would be designed to incorporate natural terrain where possible to promote aesthetics and decrease environmental disturbance. Plate 8 shows a plan view of this routing alternate.

An emergency spillway, designed to safely pass flows exceeding the 0.5 PMF without overtopping or causing damage to the dam, could be constructed with this system. The spillway would be located opposite the control structure and outflow channel to prevent interference during use. A conceptual plan of a spillway located at the west abutment is shown on the plan. The spillway would be constructed such that flows are carried past the toe of the dam before release in order to prevent damage to the dam.

Water Level at 2910' (Alternate IIb): The fine tailings in the impoundment are saturated, unconsolidated, and have little bearing capacity making standard construction methods and equipment difficult to use. Due to the expected difficulties associated with constructing the inflow channel in the fine tailings, a variation of this alternate was investigated. To reduce the problems of construction in the tailings materials, a low level dike of cohesive (low permeability) material would be constructed across the tailings impoundment, approximately 500 feet from the face of the tailings dam as recommended in the Harding Lawson Associates dam stability report. Located at this distance from the dam the potential impact of standing water on dam foundations is minimal in the judgement of engineers at Harding Lawson Associates. Top of dike elevation would be approximately 2912.0, with the water level in the impoundment maintained at 2910.0, which has been selected as the maximum practical elevation at which water can be maintained in the impoundment without significant loss of storage capacity or increasing the risks associated with saturated tailings dam foundations and sudden failure or breaching of the dike. By raising the water level in the impoundment, the length of inflow channel and subsequent tailings excavation would be reduced and this would reduce construction costs. This alternate provides water cover for much of the tailings and thereby reduces the potential for dust production and also reduces the areal extent of required revegetation. Plate 9 shows a plan view of this alternate.

There are some additional risks with this alternative, however. Should the dike leak, which it may very well do because of the difficulty in getting good compaction of the dike materials on top of tailings and the potential for seepage through the tailings material itself, a drainage channel would probably be needed below the dike. Also, in the event of a major runoff event, one slightly greater than a 100-year storm, the dike would be overtopped resulting in damage to it and to the drainage channel below the dike.

East Abutment Outlet (Alternate IIc): Placing the control structure and outflow channel on the east abutment, and the emergency overflow channel on the west abutment, as shown in Plate 8, is judged to be the best overall alternate for routing floods through the vermiculite tailings impoundment while maintaining structural integrity. Placing the outflow on the east abutment provides the most aesthetically pleasing alternate for returning the flows to Rainy Creek, with the least environmental disturbance of considered alternatives.

The east abutment area can easily be modified to construct the outflow channel without significantly disturbing the area. The outflow channel would be armored with a rock rip-rap lining and integrate a series of drop structures placed to take maximum advantage of the terrain. A natural drainage would be incorporated into the final design to increase aesthetics, and decrease excavation and construction costs.

The emergency spillway, if provided, would be constructed in natural material adjacent to the tailings dam on the west abutment to the extent that it did not interfere with the existing Forest Service road. The area is presently disturbed from mining activities. To protect the toe of the dam, the spillway will carry the flows past the toe before release. The excavated material would be placed in the groin of the dam for additional protection.

The primary disadvantage of this alternate is the longer inflow channel in the tailings, resulting in higher construction costs to excavate and construct the channel. Some drilling and blasting may be required to construct portions of the outflow channel as well.

West Abutment Outlet (Alternate IId): Locating the outflow control structure and channel on the west abutment, and the emergency spillway on the east abutment was investigated as an alternate for returning flows to Rainy Creek downstream of the tailings dam. No plans are provided for this alternate.

The primary advantage of this alternate would be to shorten the inflow channel through the tailings, reducing the extent of specialized construction to build the channel. Because the tailings are not as deep on this side of the impoundment, both the length of the channel excavation and the quantity of material to be removed would be reduced.

The primary disadvantage is the steeper sideslopes making construction of the outflow channel more difficult, and with questionable long-term geotechnical stability. A concrete drop chute (at considerable cost) or significant excavation of the abutment area may be required. Placing the emergency spillway on the east abutment would require relatively more excavation, partially in undisturbed forest, to get the flow past the toe of the dam before releasing it, reducing visual aesthetics as well. A partial relocation of the Forest Service access road would be required. Due to these engineering and aesthetic draw-backs, and lack of discernable advantages, this alternate was eliminated.

<u>Outlet Over Dam Face (Alternate IIe):</u> Constructing an outlet through the center of the dam and down the face was investigated as an alternate for returning flows to Rainy Creek. This alternate would consist of a straight inflow channel through the fine tailings connected to a reinforced concrete control structure and drop chute. Plate 10 provides a plan view of this alternative.

Placing an outlet in the dam face eliminates the need for excavation of either abutment, unless an emergency spillway is desired. The outlet control structure and drop chute would be built as one structure, and tied directly into the existing channel below the dam, eliminating the need for extensive downstream work. Overall, environmental disturbance is negligible.

There is an increased possibility of tailings dam saturation and seepage with this option. The zone of influence from the channel will affect a larger area than if it were located adjacent to an abutment. As with the other alternates, lining the channel would eliminate the problem. Long-term geotechnical stability of this system may be questionable, and construction would be moderately difficult on the steep slope.

Other disadvantages are reduced aesthetics, higher construction costs (reinforced concrete) and public safety (straight-walled drop chute and high velocities eliminate any chance of escape).

4.3.2 Evaluation of Alternatives for Channel Reconstruction in the Tailings

<u>Safety:</u> Routing floods through the tailings impoundment provides the best method to <u>safely</u> pass storm events of 0.5 PMF or larger while assuring the integrity of the dam. This concept takes advantage of the temporary storage capacity of the impoundment to reduce outflows while providing safe, effective flood routing.

The existing tailings dam is geotechnically very stable, having been designed to withstand earthquakes of a recommended magnitude with no loss of integrity. Temporarily storing peak flows provides a way of assuring minimum risk to the dam. Elimination of upstream diversion dams associated with the other main alternatives reduces risks associated with diversion dam failure.

Because of the storage capacity in the reservoir, and the emergency spillway, risk from debris/plugging is minimal for this alternative. In addition, several low maintenance structures would be installed to prevent debris from entering the control structure. During peak events, the entrance into the control structure will be submerged to prevent debris from entering into the control structure.

Reduced peak outflows will result in a considerably smaller outflow channel, making escape from the channel easier, hence better for public safety. In addition, the reduced outflows result in less flood damage to downstream structures, such as the highway.

Water Quality Impacts: With this alternate, tremolite fibers from the coarse tailings dump, and fine tailings impoundment will continue to impact surface water. However, during normal flow conditions the low gradient of the reconstructed channel and the placement of protective cover in the reconstructed channel will greatly reduce the risk of tremolite entrainment. Also it is anticipated that entrainment will continually decrease as vegetation becomes established and stabilizes the dump, impoundment, and other disturbed areas. Preliminary data from water monitoring programs indicate that water quality degradation from other mineral constituents is minimal at this site.

As discussed in Section 4.2, tremolite fibers will not be eliminated from Rainy Creek, regardless of the alternative selected. Fibers from the headwaters of Fleetwood Creek, from Carney Creek, and in the Rainy Creek streambed downstream of the impoundment, will continue to contribute to fiber counts in Rainy Creek.

Environmental Impacts: Environmental disturbance will be minimized with this alternate, especially when compared with full diversion. Some disturbance will occur during construction of the outflow channel. By reducing outflow volumes, erosion and other flood-related problems will be diminished.

Tailings/Dam Saturation: Saturation of the tailings dam in the immediate vicinity of the inflow channel, and resulting embankment stability should the toe-drains become inoperable, is a primary regulatory concern. Because of the low permeability of the fine tailings relative to the dam material, major water loss through infiltration is not expected to be as severe of a problem as with the diversion channels. Further, the rate of water movement through the fine tailings is significantly slower than the dam, as demonstrated by the piezometers installed in the dam face. Water entering the dam from the tailings or channel is expected to drain relatively quickly, hence reducing the possibility of saturation and subsequent seepage.

As discussed earlier, diverting flows to the side of the impoundment will not eliminate the possibility of tailings dam saturation. The only sure method of eliminating the risk, from any alternate, is with an impermeable channel or pipeline. Should tailings dam saturation become a problem, construction of an engineered toe drain will be completed by W.R. Grace.

Sedimentation: Increased sedimentation from the tailings impoundment is expected for a short period of time (estimated at 2 to 5 years) following closure. After that, vegetation will become established and provide slope stabilization, reduced erosion, utilization of excess water, and wildlife forage. A detailed description of re-vegetation is provided in Section 5.7 Sedimentation associated with channel excavations and other construction activities may also occur for a short time period, but will be negligible compared with a full diversion alternate.

Routing the surface water flows through the impoundment will take advantage of the remaining wetland to improve water quality through natural filtration and settlement.

In summary, routing floods through the existing tailings impoundment with a controlled outflow system provides the best method to safely control flood events meeting or exceeding the required 0.5 PMF design. This general concept provides a feasible method to safely route floods while minimizing environmental disturbance and maintenance, and improving aesthetics.

Advantages (Table 4.4) and disadvantages (Table 4.5) of routing the flood flows in a reconstructed channel through the tailings impoundment follow:

Table 4.4. Advantages associated with routing floods through the tailings impoundment.

ALTERNATE	ROUTING THROUGH IMPOUNDMENT - ADVANTAGES
All Alternatives	• Provides a higher level of public safety than other alternatives while retaining a relatively simple design; • Provides a safe, cost effective method to handle storm flows while maintaining long-term integrity of the dam; • Geotechnically the most stable alternative; • Plugging/debris problems less critical or likely; • The system is capable of handling floods larger than 0.5 PMF with the addition of a relatively simple emergency spillway; • Outflow channel relatively small, making construction feasible and cost effective; • Limited environmental disturbance; • More natural/aesthetic outflow channel; • Remaining wetland provides improves surface water quality through natural filtration and settling; • Water loss to infiltration expected to be minimal; • Less overall maintenance; • Reduced potential for airborne particulate; • Reduced outflows will reduce downstream impact from flooding.
Water Level at 2904' (Alternate IIa)	• Maintains water away from the dam face as much as possible for maximum safety.
Water Level at 2910' (Alternate IIb)	• Maintains water away from the dam face provided seepage through or under the dike is minimal; • Reduces the requirements for construction in mucky material; • Reduces requirements for revegetation; • Maximum potential for reduction of airborne particulate from the impoundment.

ALTERNATE	ROUTING THROUGH IMPOUNDMENT - ADVANTAGES
East abutment outflow (Alternate IIc)	• Less overall environmental disturbance than west side outflow channel; • Existing terrain can be easily modified for outflow channel thereby reducing environmental disturbance; • Emergency spillway on west abutment can be constructed with a minimum of excavation and disturbance; • Highest public safety of all alternates;
West abutment outflow (Alternate IId)	• Shorter inflow channel; • Bedrock does not affect construction.
Outflow over dam face (Alternate IIe)	• Eliminates excavation of abutments for outflow channels; • Negligible environmental disturbance; • Control structure and drop structure are one structure; • Minimal downstream work required.

Table 4.5. Disadvantages associated with routing floods through the tailings impoundment.

ALTERNATE	ROUTING THROUGH IMPOUNDMENT - DISADVANTAGES
All Alternates	• Inflow channel difficult to construct in fine tailings, requiring specialized construction methods and equipment and increased costs; • Does not address tremolite fiber issue actively; • <u>Possible</u> saturation of a portion of the tailings dam; • Probable increased short-term sedimentation; • Slight risk of control structure becoming plugged.
Water Level at 2904' (Alternate IIa)	• Potentially difficult construction of a long channel through soft mucky tailings.
Water Level at 2910' (Alternate IIb)	• Dike and foundation materials may seep at a significant rate creating saturated tailings downstream of the dike, thereby defeating its intended purpose; • A major runoff event will cause the dike to be breached and repair will be required; • Reduces slightly the total storage capacity of the impoundment.

ALTERNATE	ROUTING THROUGH IMPOUNDMENT - DISADVANTAGES
East abutment outlet (Alternate IIc)	• Excavation of bedrock may be necessary to construct outflow channel; • Longer inflow channel required, unless variation is ~~selected;~~ • ~~Minor relocation of Forest Service access road required.~~
West abutment outlet (Alternate IId)	• Outflow channel difficult to ~~construct on steep side slopes;~~ • May require concrete drop chute; • Emergency spillway difficult to construct on east abutment; • Portion of the Forest Service access road requires relocation or reconstruction; • Increase environmental disturbance.
Outlet over dam face (Alternate IIe)	• Long-term geotechnical stability may be questionable; • Saturation of tailings dam more likely than with other (no diversion) alternatives; • Concrete structures increase cost; • Safety concern with vertical side walls and high velocity flows; • Most unnatural of impoundment routing alternatives.

4.4 PARTIAL DIVERSION

A partial diversion of flood flows would entail diversion dam(s) and channels designed to intercept and divert flows up to and including a selected design flow, i.e. 10-year or 100-year events, which are described in Section 4.4.1 below. Flows exceeding the design capacity of the diversion dams would be allowed to by-pass the diversion dam through a "blow-out" plug of uncompacted fill placed in an engineered spillway and be routed through the reservoir using a system similar to those in Section 4.3. The concept behind this alternative would be to provide a system that would combine the advantages of a full diversion system with the advantages of flood routing through the reservoir. A full engineering analysis of these alternates is not detailed below, as many of the issues are covered in previous sections.

4.4.1 Description of Conceptual Designs

<u>100-Year Flood Diversion (Alternate IIIa):</u> A partial diversion system would require one or more dams similar to the full diversion dams, but designed to allow higher flows to by-pass them during larger events. The smaller design flows would be diverted around the impoundment in an open channel or pipeline, returning to Rainy Creek below the tailings dam. The larger flows would be routed through the reservoir using a system similar to those in Section 4.3, which includes a constructed inflow channel and outflow channel, control structure, and emergency spillway. This alternative would virtually "double" the costs for the project, by requiring both flood routing systems to be constructed.

Designing and constructing a structurally competent diversion dam capable of diverting smaller flows while by-passing larger flows will be difficult to accomplish. As stated earlier, the tailings do not provide adequate foundation for structures, making long-term structural integrity and durability questionable. A single by-pass flood event would likely cause irreparable damage to the diversion structure due to scouring of the foundation layer. Constructing separate dams for Fleetwood and Rainy Creeks above the tailings is again an option. Regardless of the diversion dam site selection, continual maintenance would be required.

Due to the adverse conditions associated with the east side (coarse tailings, bedrock, etc.), the partial diversion channel would be constructed on the west side of the impoundment. Assuming a 10 ft. flat-bottomed channel, 2:1 maximum cut slopes, and 0.005 ft/ft gradient produces the channel section shown in Plate 11. The bottom of channel elevation would be approximately 2888 at the tailings dam. As with a full diversion channel, massive cuts would be required to construct a channel that would provide long-term service. Complete relocation of the Forest Service access road would again be required.

During a 0.5 PMF event, assuming the impoundment routing system was constructed similar to those in Section 4.3, the water level in the impoundment would rise to at least 2922, making the water level in the partial diversion channel 34 feet in depth (refer to Plate 11). Obviously, this volume of flow would be impossible to control without a structure, further increasing the cost of this system while providing limited added benefit. Lining the channel would also be recommended to prevent infiltration, geotechnical instability, and possible tailings dam saturation.

An option would be to install a pipeline to carry the partial flows around the impoundment, making the system similar to the existing Rainy Creek diversion pipeline. Continual maintenance could be expected based on W.R. Grace's experience with the current pipeline, and plugging would be a problem. A pipeline system of any kind is not recommended.

A partial diversion system would require separate outflow channels for the diversion channel, and the "backup" impoundment routing system. The outflow channel for the impoundment would be constructed as described in Section 4.3, while the partial diversion would require a drop chute or some other method of returning outflows to the elevation of Rainy Creek downstream of the tailings dam.

10-Year Flood Diversion: The partial diversion of stream flows exceeding a 10-year storm event would be virtually identical to the 100-year event. The restrictions of construction equipment dictate that the diversion channel would assume basically the same dimensions. The only significant design variation is in the outlet control structure from the diversion dam(s) which needs to be more restrictive in order to limit flow. The smaller outlet, is a potential source of problems in that will be more subject to plugging by debris and will likely require more frequent cleanout.

One perceived advantage of this alternate is the periodic wetting of the tailings which might be beneficial for maintenance of vegetation and reduction of potential dust production. However, this wetting would be incomplete at best and its benefits would be questionable on such an infrequent and unpredictable basis.

4.4.2 Evaluation of Partial Diversion Alternatives

Safety: From a safety standpoint, partial diversion does not improve safety over the "no diversion" alternate, however, it is significantly better than a full diversion system. The reasons are covered in previous sections. Plugging or failure of smaller partial diversion dams would be less critical.

Water Quality Impacts: Asbestiform fiber contamination of surface water from the tailings impoundment would be reduced by diverting the "day-to-day" smaller flows around the impoundment, but would not be eliminated as discussed in Section 4.2.

Environmental Impacts: The environmental disturbance would be the most significant of any option. Massive excavations would be required for the diversion channel and drop chute. All excavation required for the outflow channel associated with routing through the impoundment would remain as well. Downstream impact would be reduced when compared to full diversion, but would be greater than the alternates routing floods through the impoundment.

Tailings/Dam Saturation: The possibility of saturating a portion of the tailings dam due to continuous flow through the impoundment will be eliminated, however, saturation from the diversion channel remains a possibility unless channel lining is installed.

Sediment: Short-term sedimentation from the tailings impoundment would be reduced with this alternative, but may increase from the major excavations associated with the diversion channel. The advantage of using the impoundment wetland for improving surface water quality through natural filtration and settling would be eliminated.

In summary, using a partial diversion system in conjunction with an impoundment routing system does not increase safety over the impoundment routing system. This alternate greatly increases costs. Maintenance and environmental disturbance increase, and geotechnical stability, construction feasibility, tailings dam saturation, and sedimentation remain as issues.

Advantages (Table 4.6) and disadvantages (Table 4.7) of the partial diversion alternate are summarized below:

Table 4.6 **Advantages associated with partial diversion flood routing systems.**

ALTERNATE	PARTIAL DIVERSION - ADVANTAGES
All Alternates	• Will provides a higher level of public safety than a full diversion; • Geotechnically more stable than full diversion; • Plugging of channel from flood debris less critical than full diversion; • Possible reduction in downstream tremolite fiber concentration in surface water; • Possible reduction in short-term sedimentation from the tailings impoundment.
100-Year Flood Design Basis (Alternate IIIa)	• Diversion dam outlet structures will be less prone to plugging than those for a 10-year flood.
10-Year Flood Design Basis (Alternate IIIb)	• Periodic wetting of tailings <u>may</u> enhance growth of vegetation and provide for some degree of dust control; • Marginally lower costs for channel lining materials.

Table 4.7 **Disadvantages associated with partial diversion flood routing systems.**

ALTERNATE	PARTIAL DIVERSION - DISADVANTAGES
All Alternates	• Adds no safety benefit to impoundment routing (no diversion) alternative; • Partial diversion dams difficult to construct in fine tailings, requiring specialized construction methods and equipment and increased costs; • Long-term stability and integrity of partial diversion dams questionable; • Increases overall cost of the project significantly due to combination of systems; • Increased maintenance, particularly with partial diversion dams; • Saturation of tailings dam remains a possibility without diversion channel lining; • Possible increased short-term sedimentation from excavation; • Does not take advantage of impoundment wetland; • Plugging of smaller partial diversion channels; • Largest environmental disturbance of all alternatives.

ALTERNATE	PARTIAL DIVERSION - DISADVANTAGES
100-Year Flood Design Basis (Alternate IIIa)	• Tailings will not receive a thorough wetting on any reasonably short time frame.
10-Year Flood Design Basis (Alternate IIIb)	• More prone to plugging than a system designed for larger flows.

4.4 SUMMARY/CONCLUSIONS

Based on findings of the engineering analysis of the various flood routing alternatives, routing the flood through the tailings impoundment using a designed structure to control discharges to an east abutment outflow channel appears to be the best, most feasible, and safest method for flood routing Rainy Creek through the vermiculite tailings impoundment area. In our judgement, safety should be the overriding factor in selection of a permanent reclamation plan. This alternate provides sufficient storage capacity within the impoundment to receive a 0.5 PMF event without utilizing an emergency spillway which is provided in the event of an even larger storm.

The recommended alternative does violate a provision of the permit requiring diversion of water around mine wastes at closure. This could be a matter of concern from the standpoint of water quality issues. It should be understood that this mine is not a base metal mine and does not produce acid mine drainage typically containing high levels of metals. One significant area of potential concern is tremolite fiber entrainment. However, Rainy Creek is not utilized directly as a drinking water source. Other alternatives will not totally eliminate this concern since Fleetwood Creek and Carney Creek originate in areas where natural outcropping of tremolite occurs or which have been subject to disturbance by mining activity.

In order to reduce these concerns, disturbed areas will be stabilized to reduce erosion through the establishment of vegetative cover. Similar measures are proposed for the tailings impoundment to reduce the level of suspended particulate in surface waters discharged through the dam. Included in these measures will be revegetation of tailings beach areas and installation of channel linings to stabilize the channel and prevent direct contact with underlying tailings material. A program to establish current water quality levels is underway and will continue on a regular basis as reclamation proceeds. Overall entrainment of asbestiform fibers from the tailings should be minimal due to these design measures.

Another area of concern may be the establishment of a precedent for reclamation by allowing surface waters to be routed through mine waste facilities. Had this facility presented a significant risk to water quality, our recommendations would have been entirely different. As it is, the resolution of the safety and long-term stability aspects of the existing situation appear to take precedence over the relatively minor water quality issues, which are not life threatening or environmentally damaging. In summary, site-specific considerations make channel re-establishment a sound decision where at other facilities diversion may be more technically sound.

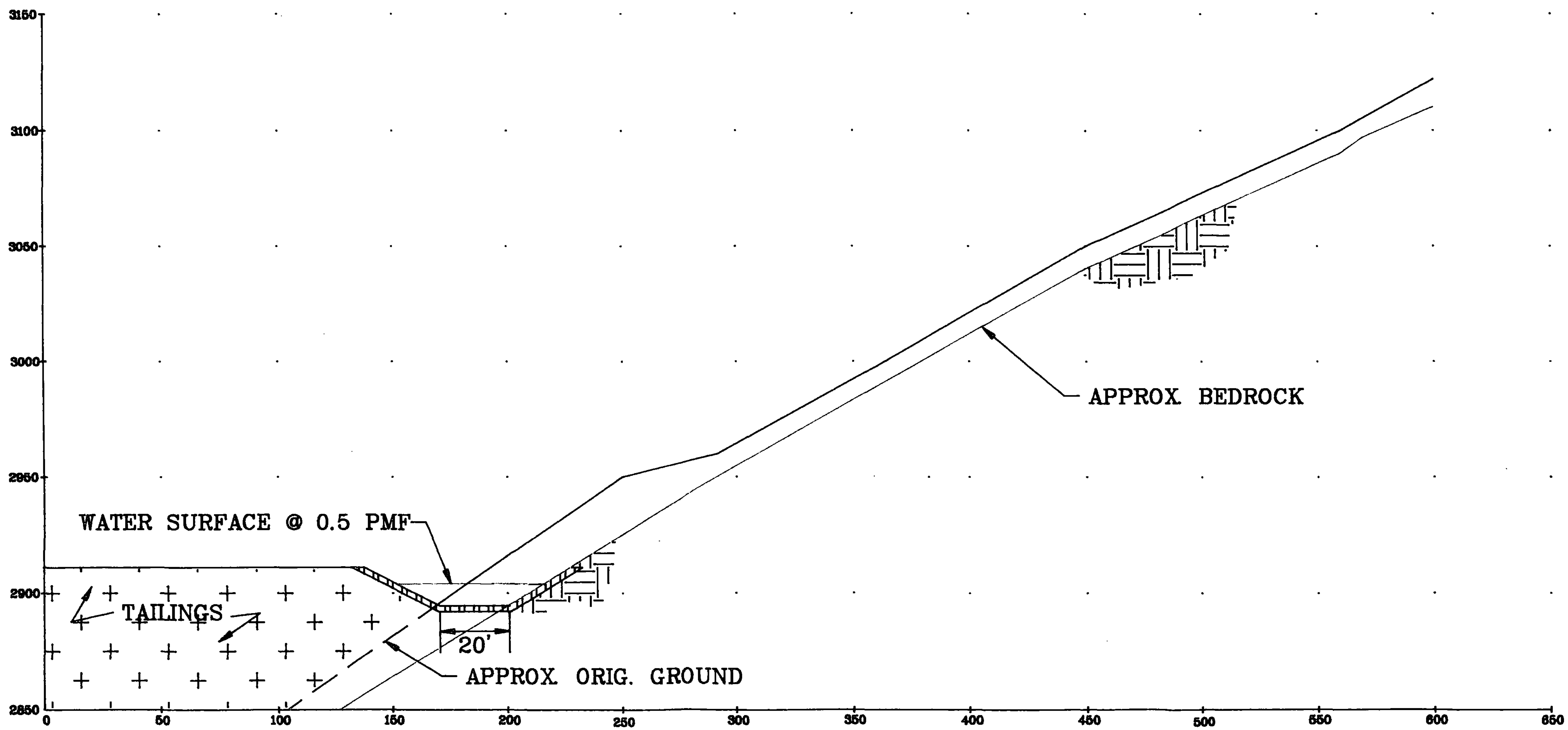


Plate 6. **Typical section of the east side full diversion channel constructed partially within the fine tailings impoundment.**

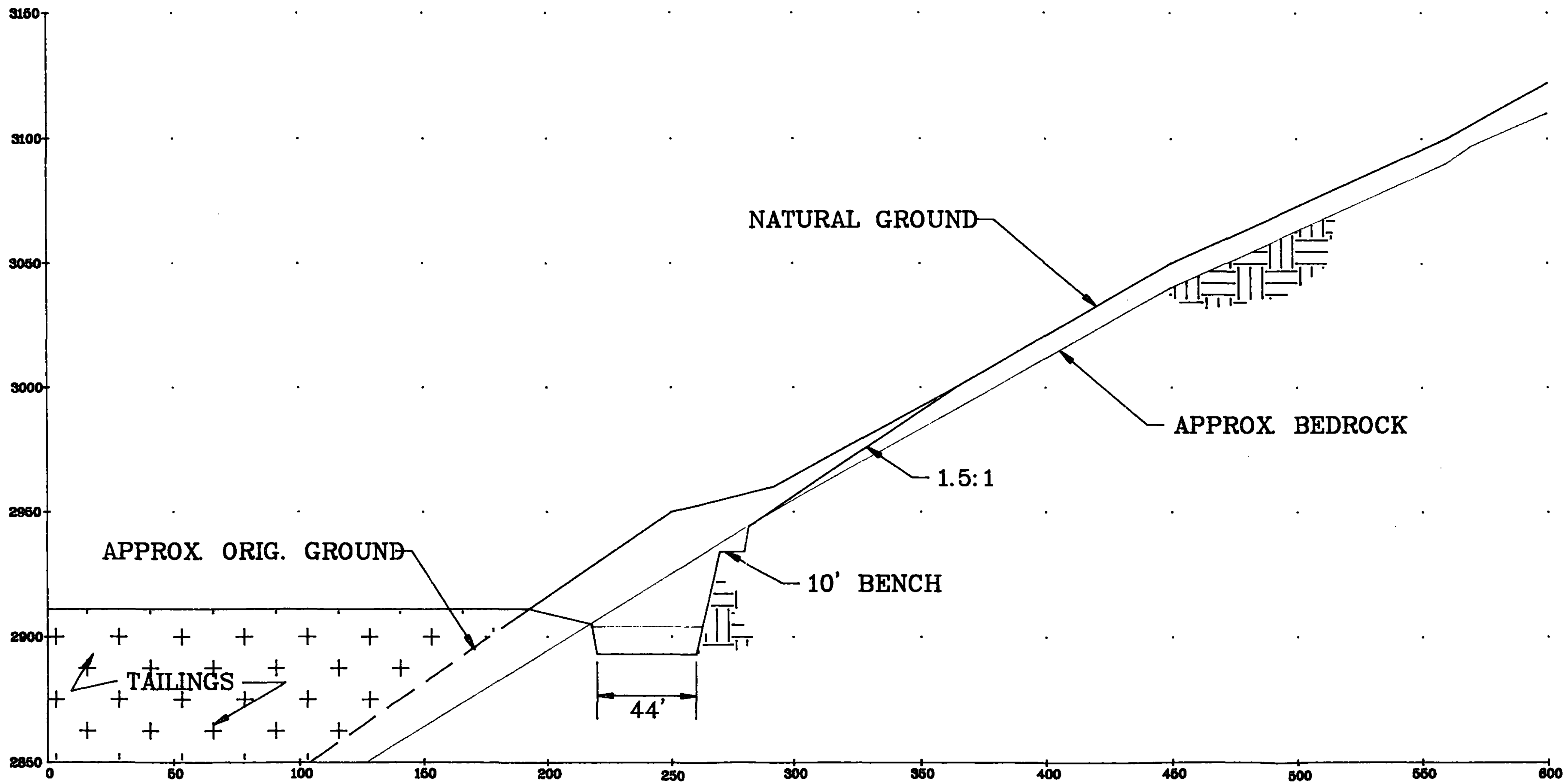


Plate 7. **Typical section of the east side full diversion channel constructed in bedrock, outside of the fine tailings.**

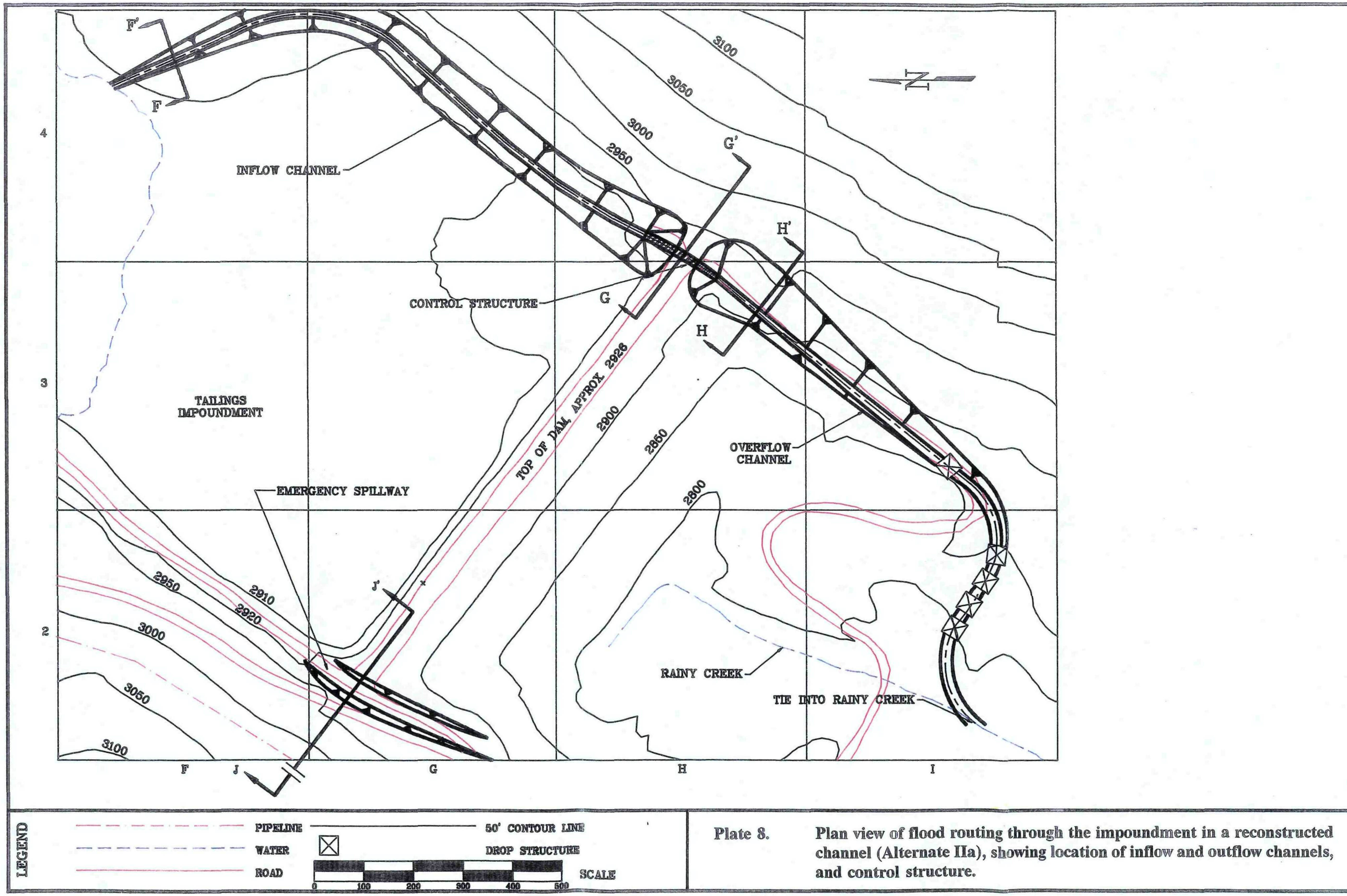
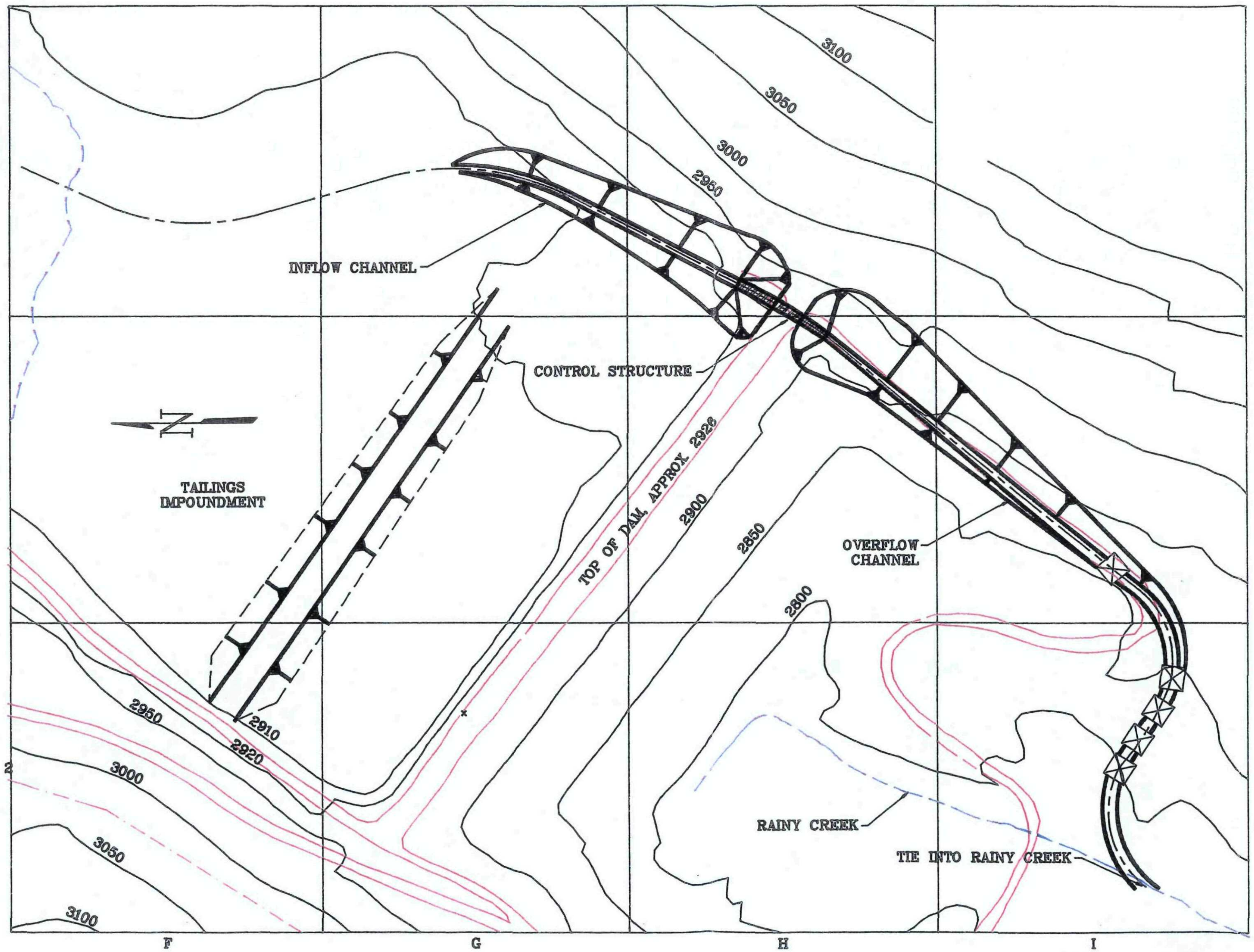


Plate 8. Plan view of flood routing through the impoundment in a reconstructed channel (Alternate IIa), showing location of inflow and outflow channels, and control structure.



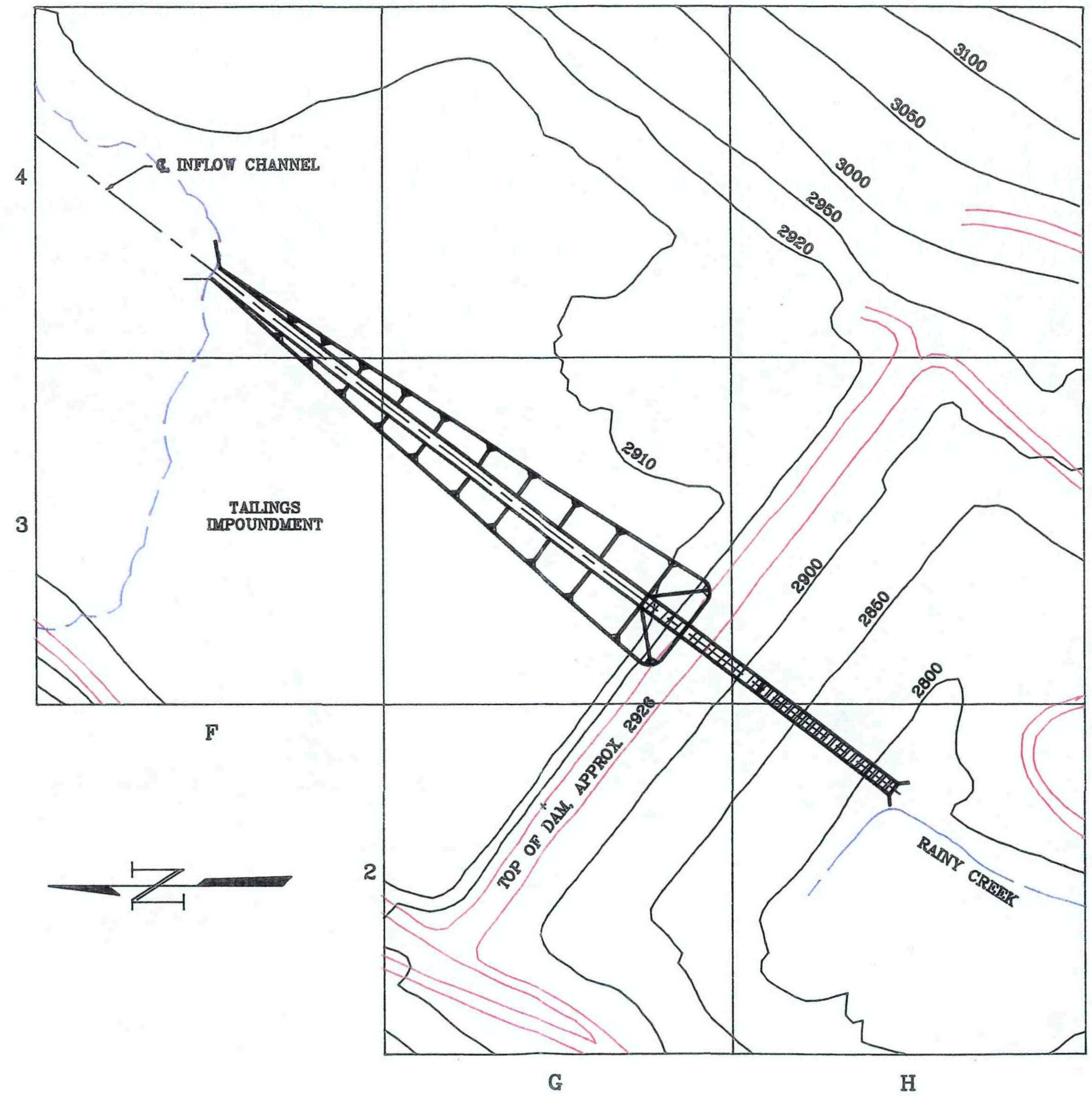
LEGEND

PIPELINE — 60' CONTOUR LINE

WATER — DROP STRUCTURE

ROAD — SCALE 0 100 200 300 400 500

Plate 9. Plan view of flood routing system through the impoundment with low profile dike (Alternate IIb) showing the dike and revised inflow channel. Other structures are the same as Plate 8.



LEGEND	
PIPELINE	50' CONTOUR LINE
WATER	
ROAD	SCALE 0 100 200 300 400 500

Plate 10. Plan view of outlet/control structure over dam face for routing floods through the tailings impoundment (Alternate IIe).

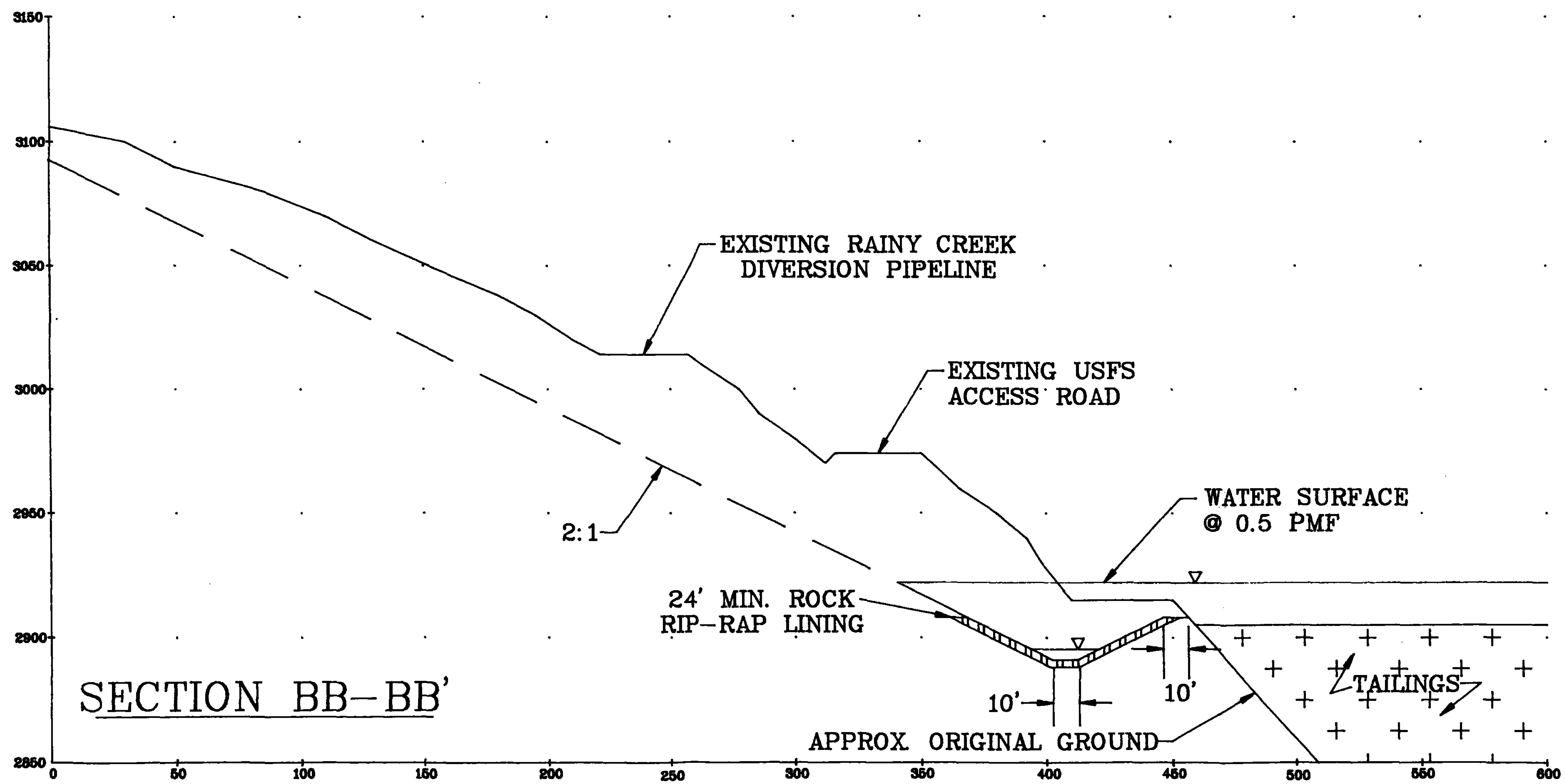
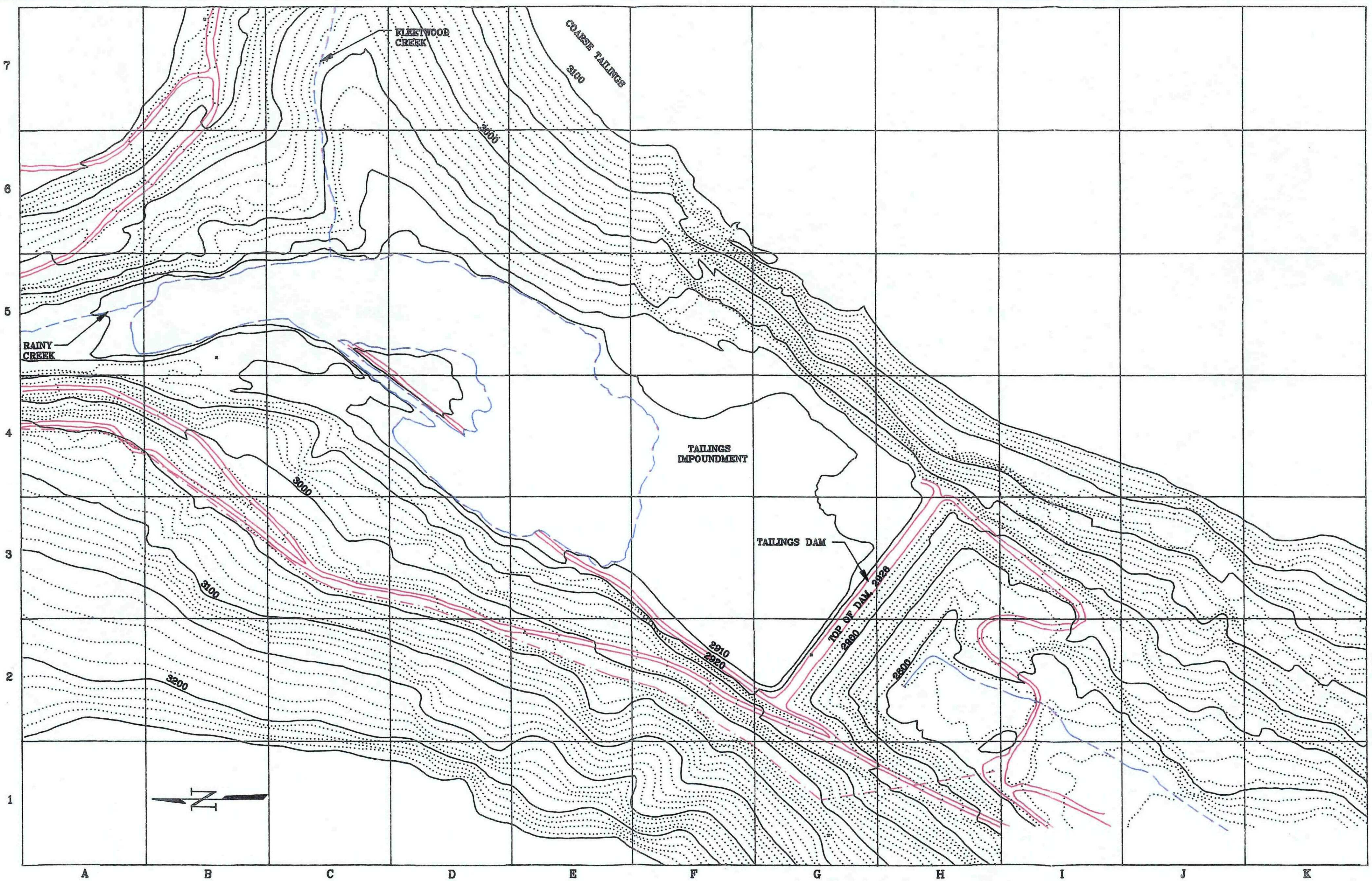


Plate 11. Typical section for a partial diversion channel at west abutment area.

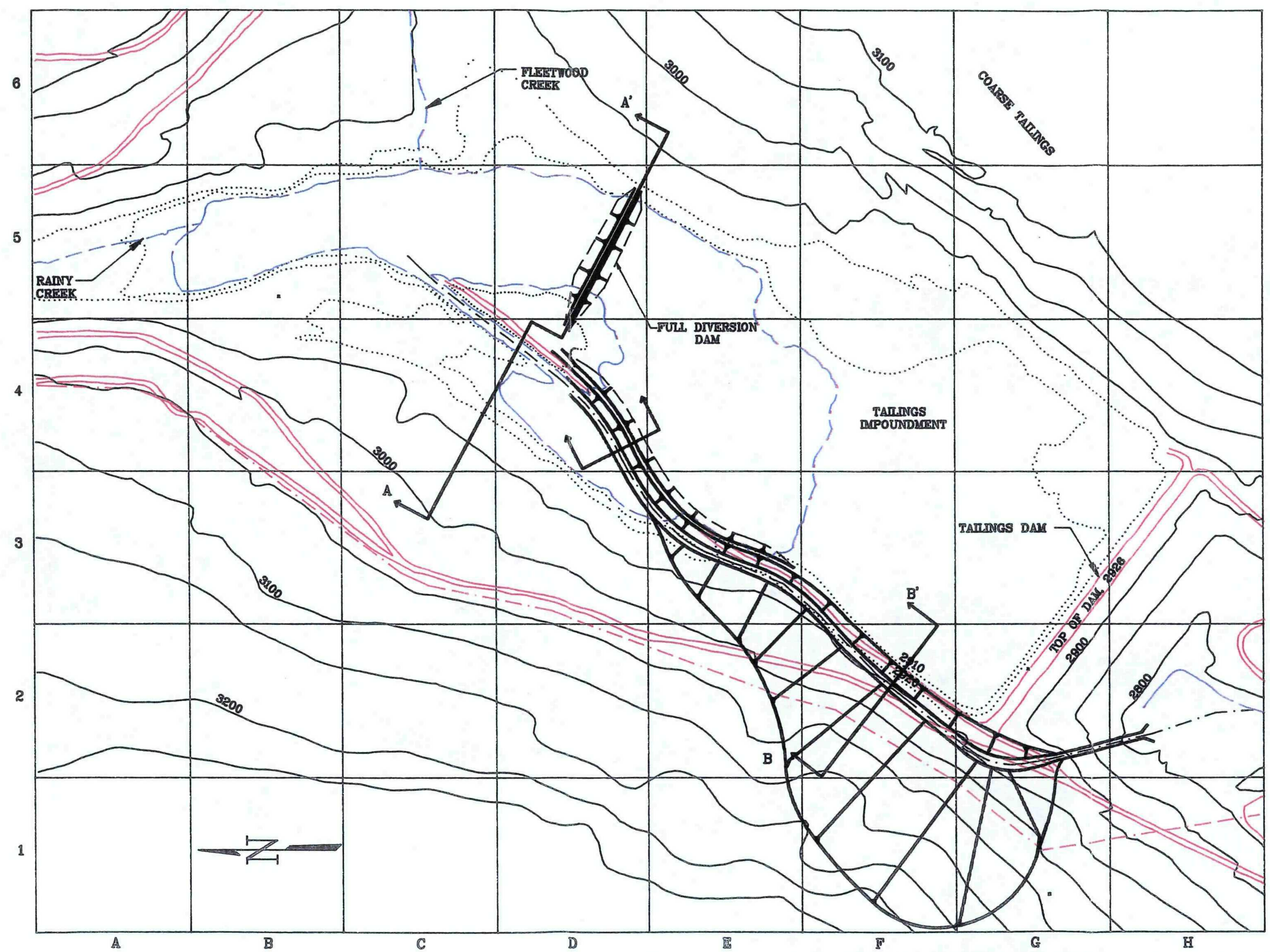


LEGEND

PIPELINE
WATER
ROAD
50' CONTOUR LINE
10' CONTOUR LINE
SCALE 0 100 200 300 400 500 600 700 800 900 1000

REFERENCE	REVISION

Plate 1. Plan view of project area



LEGEND

PIPELINE — 50' CONTOUR LINE
WATER — 10' CONTOUR LINE
ROAD — SCALE 0 100 200 300 400 500 600 700 800 900 1000

Plate 2. **Plan view of conceptual full diversion flood routing system employing a single diversion dam below the confluence of Rainy Creek and Fleetwood Creek. Cross-section typical of diversion dam construction indicated by Section A-A'.**

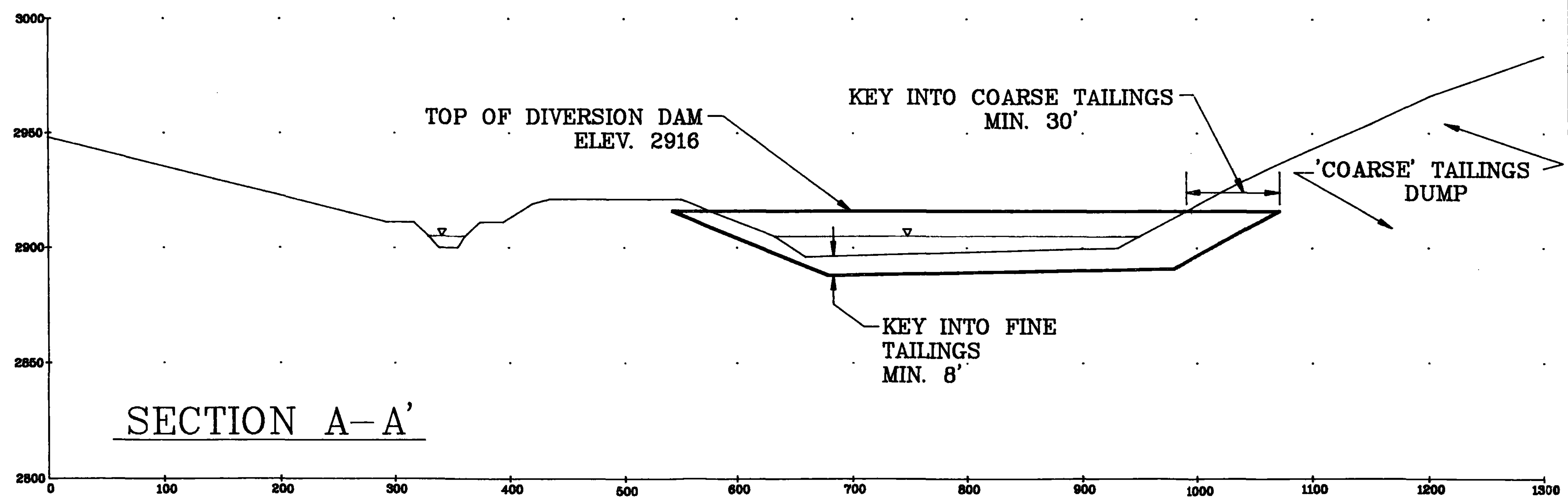
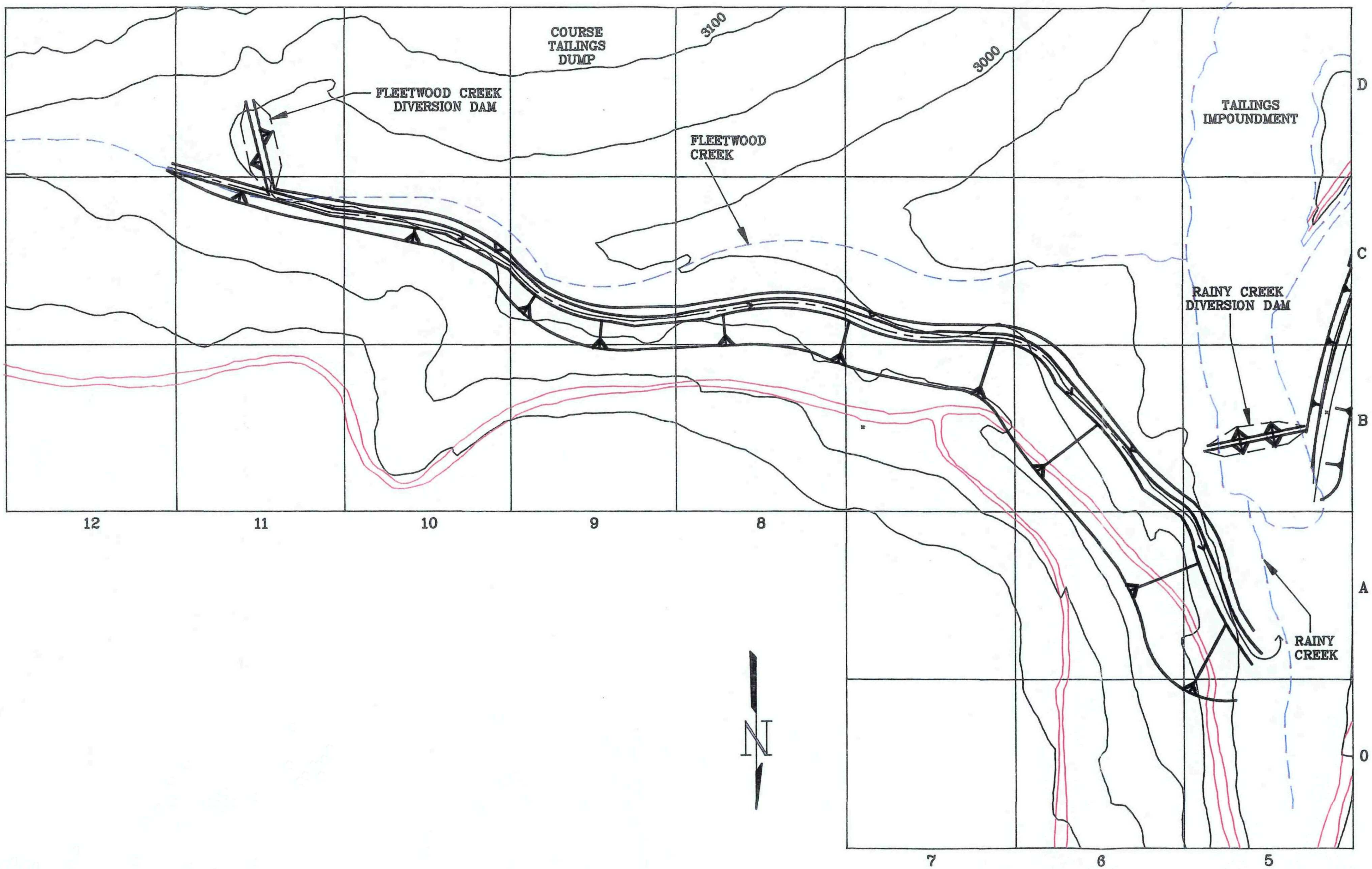


Plate 3. **Cross-section A-A' showing typical section of diversion dam for a full diversion flood routing system.**



LEGEND

PIPELINE
WATER
ROAD
50' CONTOUR LINE
SCALE
0 100 200 300 400 500 600 700 800 900 1000

Plate 4-A. Plan view of full diversion dam and channel to deliver Fleetwood Creek to the Rainy Creek diversion dam. Diversion dam located above coarse tailings dump. This figure to be matched with Plate 4-B.

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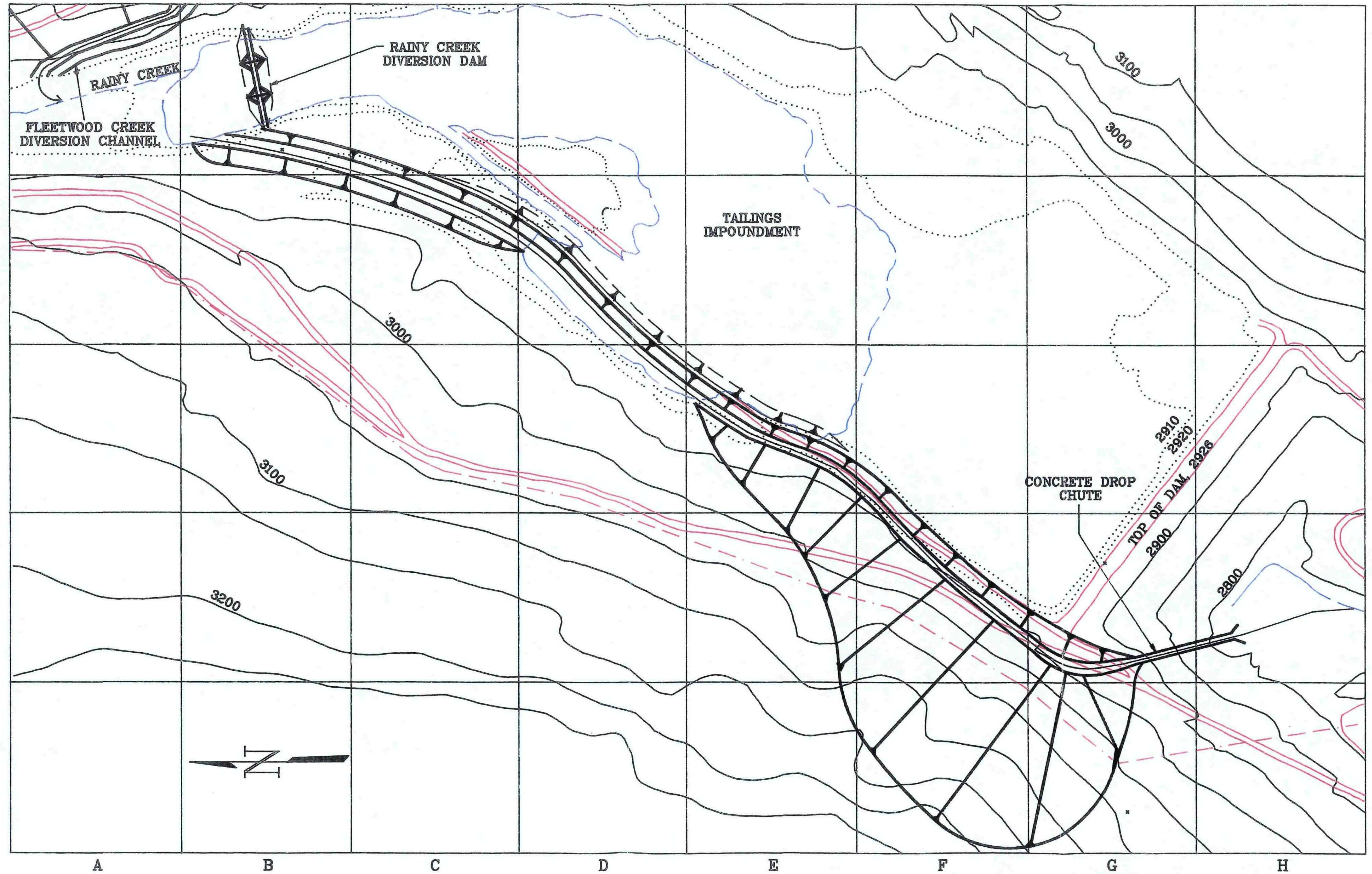

LEGEND

PIPELINE — 50' CONTOUR LINE

WATER — 10' CONTOUR LINE

ROAD — SCALE 0 100 200 300 400 500 600 700 800 900 1000

Plate 4-B. Plan view of Rainy Creek diversion system employing a dam upstream of tailings. Fleetwood Creek diversion channel enters from the east. This figure to be matched with Plate 4-A.

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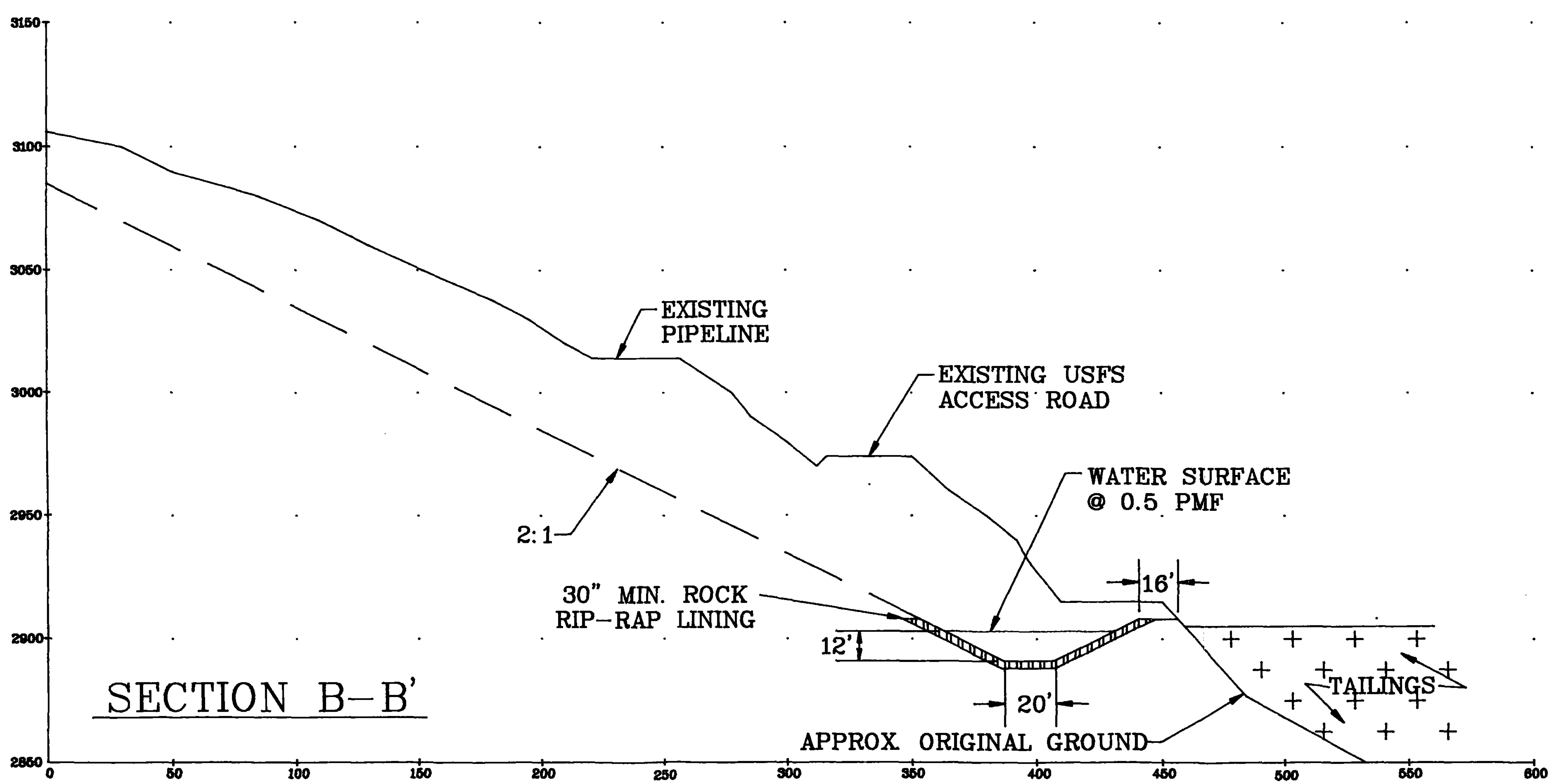

Plate 5. **Typical cross-section of west diversion channel showing limits of excavation.**

LSB 66 00058

5.0 PROJECT DESIGN - PREFERRED ALTERNATIVE

5.1 GENERAL DESIGN APPROACH

The general design approach of the preferred alternative entails routing all flood flows from both Rainy Creek and Fleetwood Creek into the tailings impoundment, controlling discharge with a control structure, and returning water to Rainy Creek (downstream of the tailings dam) by means of an outflow channel. No diversion structures will be employed. Flows in excess of 0.5 PMF will be handled with an armored emergency spillway. Routing flows through the impoundment provides the safest method of passing major flood events through the impoundment area, while maintaining the long-term integrity of the tailings dam. The advantages of such a system have been demonstrated in Section 4.0.

Flood flows enter the impoundment unrestricted and, depending on the discharge rate, are passed directly through the impoundment and discharged, or temporarily stored in the existing reservoir until discharged. Discharges from the impoundment are restricted to a design peak outflow by means of a concrete box culvert. Discharges from the control structure enter a constructed outflow consisting of a rock-lined, trapezoidal channel connecting a series of concrete drop structures. Flows are returned to Rainy Creek approximately 800 feet below the tailings dam. An inflow channel will be constructed in the tailings in order to connect the impoundment wetland with the control structure. This system will allow W.R. Grace to maintain a relatively constant water surface elevation (in the wetland) to aid in revegetation, and prevent saturation of the tailings dam.

Other work proposed during closure includes removal of the existing water control structures (Rainy Creek diversion, emergency spillway, and decant tower); providing a stabilized Fleetwood Creek channel through the coarse tailings dump; revegetation and other erosion control and surface stabilization measures; and, general reclamation efforts to improve natural aesthetics of the impoundment area.

Plate 8 shows a plan view of the tailings impoundment with the preferred flood routing alternate overlain. Following sections provide greater detail of the proposed closure plan for the vermiculite tailings impoundment.

5.2 TAILINGS IMPOUNDMENT

The tailings impoundment will basically remain as it currently exists with a pond and associated fringe of emergent vegetation (wetland), "beach" area, dam, and inflow from Rainy and Fleetwood Creeks. The flood routing system will be constructed, the existing water control structures removed, and revegetation/reclamation work completed during closure.

Following closure, the pond will be retained as a natural wetland. The wetland will have a water surface elevation of 2904 ± and will encompass approximately 20 acres in the middle to upper portion of the impoundment. Water depths will range from 0 ft at the water's edge to a maximum of about 7 ft, with an average depth estimated at 2 to 4 feet. The water's edge will remain approximately 700 to 800 feet from the dam creating a "beach" area (between the water's edge and the upstream face of the dam) of slightly less than 20 acres. Revegetation will take place on the entire impoundment area (see Section 5.7). The estimated boundaries of the wetland, following closure, are represented by blue lines on Plate 8.

Inflow from Rainy Creek will continue to enter the impoundment from the north. The (Rainy Creek) diversion structure, located approximately 1 mile upstream of the impoundment, and associated pipeline together with the present emergency spillway and decant tower/pipeline will be removed. Fleetwood Creek will be restored to a stabilized channel located adjacent to the toe of the coarse tailings dump, and enter the impoundment from the east. Neither flow will be restricted or diverted.

A flood routing control system for the impoundment will be constructed on the lower (dam) end. Details are located in following sections.

5.3 INLET CHANNEL

An inlet or inflow channel, from the edge of the wetland to the control structure, will be constructed as part of the preferred flood routing system. In addition to flood routing, the inflow channel will provide passage for low flows through the impoundment to prevent the water surface elevation in the pond from rising, inundating the beach area, and eventually saturating the tailings dam. The inlet channel is shown on Plate 8.

The inflow channel will connect the wetland with the control structure. The channel crest elevation (at the edge of the wetland) will be set at 2904.0 ±, and the crest elevation of the control structure will be set at 2900.0, making a channel gradient of approximately 0.0038 ft/ft or 0.38%. Maximum calculated flow velocity in the inflow channel will be 5.5 feet per second. Plate 12 represents a section following the centerline of channel, identifying elevations, grades, etc. for the inflow channel, control structure, and outflow channel.

.38 %

The inflow channel will be a trapezoidal construction with 10 ft wide bottom, and a combination of 2:1 and 3:1 sideslopes. Plate 13 shows a typical inlet channel cross-section. The bottom and sides of the channel (to 7 ft elevation) will be covered with a non-woven geo-textile, followed by a 6 inch bedding layer of "dirty" gravel, and overlain by a 12 inch (minimum) layer of well graded D_{50} = 4" cobbles with fines (dirty) and seeded. In addition to providing bedding for the cobble channel lining, the dirty gravel will improve revegetation success in the channel, and substantially reduce the contribution of tremolite fibers from that portion of the channel. The dirty cobble lining should also improve reclamation success, further stabilizing the channel against storm events. The channel lining will be keyed into the sides of the channel as shown.

3:1 Probably

The lined portion of the channel will be excavated at a 2:1 slope, with the upper portions excavated at a 3:1 (refer to Plate 13). The concept behind this design is that the upper, unlined portions of the slope will have less potential for erosion prior to vegetation becoming established with a flatter slope. Also, vegetation will have a better success rate, and will become established quicker. Armoring the 2:1 slopes will prevent erosion and flood scour from occurring until vegetation becomes established. Should slope stability or other problems become evident during actual construction, the slopes will be flattened at that time.

5.4 CONTROL STRUCTURE

A control structure will be constructed through the tailings dam to control discharges from the reservoir, and into the outlet channel below the dam. The control structure will provide a method for safely reducing peak flows during major events while preserving the integrity of the dam and reducing the downstream impact. Our study of various control structures including open channels, concrete box culverts and metal pipe culverts suggests that the concrete box culvert provides the best method for controlling outflow while preserving the surge capacity of the impoundment for major storm events.

For the purpose of the conceptual study, we investigated two configurations for the box culvert control structure. These were twin 4 ft. by 6 ft. concrete box culverts (total open area 46.6 square feet), and a single 4 ft. by 8 ft. concrete box culvert (total open area 31.4 square feet). Both structures have an inflow elevation of 2900.0, and a 2% grade. Entrance construction will match adjacent contours.

Calculated peak outflow (26 feet elevation head) from the twin box culverts is 1080 cfs, and 744 cfs from the single box culvert. Design calculations for peak outflow are located in Appendix C. We then looked at the performance of these outlet structures under several flow conditions including the 100-year storm event and the 0.5 PMF event. A discussion of the performance of the systems under these conditions follows in Sections 5.4.1 and 5.4.2. Pertinent findings of this analysis are summarized in Table 5.1.

5.4.1 100-Year Event

Routing the 100-year, 24-hour event peak inflow of 460 cfs (Section 3.2.2) through the reservoir using a crest elevation (beginning of the inflow channel) of 2904.0, and the twin box culverts for outlet control, produced a peak discharge of 228 cfs and a maximum water surface elevation of 2903.8 at the outlet control structure. This demonstrates that the surface water elevation of the impoundment will not rise significantly above the elevation of 2904.0 during a 100-year event if the twin box culverts are used to control outflow.

This is an important point. Should a 100-year event occur immediately before a 0.5 PMF event, the surface water elevation in the impoundment will remain at the proposed static water elevation of 2904.0. Because of this, routing/storage will begin at elevation 2904.0 rather than the existing emergency crest elevation of 2920.0 as outlined by the Montana Dam Safety regulations (State of Montana, 1989).

5.4.2 Probable Maximum Flood

Various percentages of the PMF event, beginning with a minimum of 0.5 PMF, were routed through the impoundment using representative outlet control scenarios, including with and without an emergency spillway. The results of the flood routing models are located in Appendix C.

Routing 0.5 PMF using the twin box culvert control (no emergency spillway) produced a peak discharge of 983 cfs, and a maximum water surface elevation of 2921.95, or slightly over 4 feet of freeboard remaining at the worst case. This routing was modeled using an initial water surface elevation of 2900, rather than the expected elevation of 2904. Final water surface elevations are estimated to be about 0.6 feet higher than the model results, or approximately 2922.55.

Routing 0.5 PMF using the single box culvert control (no emergency spillway) produced a peak discharge of 731 cfs at a maximum water surface elevation of 2925.1, or slightly less than 0.9 feet of freeboard. Again, a beginning elevation of 2900 was used, making the peak elevation slightly higher.

An event with a peak flow of 6320 cfs (approximately 0.55 PMF) was routed through the reservoir using the twin box culverts, and no emergency spillway. This event produced a peak outflow discharge of 1078 cfs, and a peak water surface elevation of 2925.9, or approximately 0.1 feet of freeboard.

A fourth model was completed to determine what peak flow the reservoir would safely handle with a 50 foot wide (2:1 sides) emergency spillway channel in conjunction with the twin box culverts. Setting the crest of the emergency spillway at elevation 2922.0 would allow an event of approximately 7750 cfs (0.66 PMF) through the impoundment without overtopping the dam. Maximum water elevation would be 2925.9.

The proposed location of the emergency spillway imposes constraints on the amount of available space to construct the spillway without affecting the existing USFS road. In actuality, the emergency spillway will be 30 to 35 feet in width rather than the proposed 50 ft. width. It is estimated that the peak flow that could be passed through the impoundment without overtopping the dam would be (approximately) 7200 cfs. However, for comparison of alternatives, a 50 ft. emergency spillway width will be used for the models.

The preferred control structure for this conceptual design is a single concrete box culvert with dimensions 4 ft by 8 ft, and an estimated total length of 120 feet (inlet to outlet). This structure will be less prone to blockage by debris and easier to maintain because of its large open area. The control structure will be placed in the east abutment of the tailings dam adjacent to the bedrock, and graded at 0.02 ft/ft, or 2%. Inlet (crest) elevation will be 2900.0 and outlet elevation approximately 2897.6. The control structure will outflow directly into the outflow channel (Section 5.5). Plate 14 shows a typical cross-section of the control structure at the centerline of the tailings dam.

Peak inflows from large events will enter the reservoir and be temporarily stored until discharged through the control structure at a greatly reduced rate. With a peak inflow of 5838 cfs at 0.5 PMF, and a maximum control structure discharge of 744 cfs at 26 feet elevation head (distance from free water surface to control structure crest elevation), outflows are reduced by greater than 85%.

Every precaution will be taken during final design and construction of the box culvert in the tailings dam to insure against failure and maintain the integrity of the dam. The box culvert will be bedded, backfilled, and compacted following strict specifications. Rip-rap in the apron approach to the inlet of the culvert will be upgraded to compensate for the acceleration of flow as it converges on the opening of the culvert. Provisions will be made for collection of debris before the culvert entry which could be substantial in a major flood. Constant on-site supervision will be provided by a Registered Professional Engineer.

II 12. DBL Question

With the reduction in peak discharge, the outflow channel will be considerably smaller and more stable, and flood impact to downstream areas will be greatly reduced as well.

5.5 OUTLET CHANNEL

The outlet or outflow channel will be constructed as part of the flood routing system, and will carry discharges from the reservoir control structure and return them to the natural Rainy Creek channel downstream of the tailings dam. The outflow channel, constructed on the east abutment, will consist of a heavily armored channel in conjunction with a series of concrete grade control or drop structures. This type of construction will be both functional and aesthetically pleasing, and will quickly return the flows to Rainy Creek. Environmental disturbance will be kept to a minimum. Plate 8 shows the outflow channel in plan view.

The channel will begin at the outlet of the control structure (elevation 2897.6) and tie into the Rainy Creek channel at approximate elevation 2780, with a total length of about 1300 feet. Maximum gradient will be slightly over 0.04 ft/ft (4%), and will be adjusted to "fit" the existing terrain. Maximum drop height of the drop structures will be 12 feet. A section following the centerline of channel is found on Plate 12.

A typical cross-section of the of the channel will be trapezoidal construction with a 10 foot wide bottom and 2:1 sideslopes, heavily armored with a minimum of 42 inches of rock rip-rap and underlain with a sand/gravel layer or a non-woven geotextile filter cloth. The rip-rap will be well graded with a minimum size of 3 inches and a maximum size of 36

Table 5.1 **Flood routing parameters for various routing alternatives.**

DESIGN FLOOD	PEAK FLOW (cfs)	CONTROL STRUCTURE	PEAK DISCHARGE[1] (cfs)	PEAK WATER ELEVATION (ft)
100-Year	460	twin 4' x 6'	228	2904.0
0.5 PMF	5838	as above	983	2922.6
0.5 PMF	5838	single 4' x 8'	731	2925.1
0.55 PMF	6320	twin 4' x 6'	1078	2925.9
0.66 PMF	7750	as above	2071[2]	2925.9

1 Peak discharge from the proposed control structure.
2 Includes outflow from the proposed emergency spillway.

inches. A 12-foot wide access road will be constructed on the inside berm. Plate 15 shows a typical outflow section.

The grade control structures proposed will be straight reinforced concrete drop structures similar to the SCS Type C structures, with a maximum drop height of 12 feet. The drop structures will be placed to utilize existing terrain, and depending on foundation conditions encountered during final design field investigations, some modifications may be required. Approximate drop structure locations are shown on Plate 8. Appendix D contains a standard drawing for a Type C drop spillway.

Construction of the outlet will require a moderate amount of excavation in the hillside adjacent to the east abutment of the tailings dam. With the close proximity of bedrock, portions of the channel will be in weathered or unweathered bedrock, requiring drilling and blasting. Some modification of the designed sideslopes of the outflow channel may be made should final design field investigations indicate the presence of durable bedrock. The intent of the project is to align the channel to maximize the use of the existing terrain and minimize environmental disturbance.

5.6 EMERGENCY SPILLWAY

An emergency relief spillway will be constructed on the west abutment of the tailings dam, and work in conjunction with the main flood routing system to assure safe passage of storm events exceeding 0.5 PMF. It will be sized to provide additional flood routing capacity within the constraints of maintaining construction within the abutment area of the dam but without necessitating a relocation of the Forest Service road. The spillway is designed to prevent overtopping of the tailings dam for storms with peak inflows of approximately 7750 cfs or 0.66 PMF. Construction of this emergency spillway is not required by regulation, but as a method of improving public safety. Plate 8 shows the general location of the spillway.

The emergency relief spillway will be constructed adjacent to the west abutment of the tailings dam, and will terminate 300+ feet downstream of the centerline of the dam. The design will prevent damage to the dam by delaying release of the overflows until past the toe of the tailings dam.

A typical cross-section of the of the emergency relief spillway will be trapezoidal construction with a 30 to 35 foot wide bottom and 2:1 sideslopes, armored with a minimum of 36 inches of well graded rock rip-rap. Plate 16 shows a typical cross-section of the relief spillway.

5.7 REVEGETATION

Revegetation of the tailings impoundment area will stress the re-establishment of plant species for slope stabilization, reduced erosion, utilization of excess water, aesthetic enhancement and self perpetuating vegetation for wildlife. The re-vegetation plan includes grasses, forbes, shrubs, and trees.

A specific grass mix will be used for reseeding, with each specie selected for a particular advantage that will include fixing nitrogen, production of organic matter, early emergence for soil cover and species with deep root penetration to stabilize the soil and recover water from a greater soil thickness. The tailings impoundment area will be hydroseeded at approximately 24 lbs PLS/acre and 2000 lbs/acre organic mulch where soil conditions permit. The mulch will aid in erosion control, soil aeration, seed germination, seedling establishment, and organic material. Broadcast seeding will be done on the soft tailings materials which provide poor bearing capacity for hydromulching equipment. An 18-46-0 fertilizer will be applied concurrently to improve plant growth, color and vigor. All seeding will take place in the spring or early fall.

The lower, wetter portions of the tailings impoundment area are characteristic of riparian sites which naturally promote fast growing native species such as willow, aspen, alder, chokecherry, dogwood, current, serviceberry and rose wood. These species will be planted to utilize excess water on the area surrounding the tailings pond and the beach area. Larger-sized trees are subject to wind-throw and are not recommended for this specific location.

Smaller trees and shrubs will be planted along the side slopes of the tailings dam and excavated channels. Certain provisions of the dam safety law prohibit trees on the face of dams. However, since the impoundment will normally not be holding water at capacity, the use of trees to stabilize the dam face, particularly at the lower elevations, would appear to offer more benefits both aesthetically and structurally than leaving the face of the dam entirely barren. Shrubs will quickly establish a denser cover to protect tree seedlings and new grass. Roots, especially those of woody vegetation, help stabilize banks by holding soil, reduce sediment flow and increases hydraulic resistance flow.

The coarse tailings dump has already been reclaimed and revegetated. Dozer basins were installed as catchments for runoff in order to reduce the potential for erosion. The entire coarse tailings area was seeded with a mixture of grasses and clovers. Several thousand trees and native plant species have been planted randomly along the face of the coarse tailings dump and in the dozer basins.

The tailings impoundment is currently used by moose which forage for aquatic vegetation near its edges. The reestablishment of vegetation on other areas of the impoundment will encourage use by deer and elk which are also commonly seen in the area. The use of specific cultural treatments, proper seed selection and a diversity of woody plant material will aid in the re-establishment of vegetation which will have probable long-term soil stabilization and assist in the natural regeneration of a productive forest habitat.

5.8 STABILIZATION/EROSION CONTROL

An important constituent of the flood routing system, and other (tailings impoundment) closure activities will be reduction of erosion and long-term stabilization. This is particularly important at this site as the tailings impoundment and coarse tailings dump are basically devoid of vegetation at the present, making them prone to erosion and other problems. W.R. Grace will exercise best management practices to reduce these concerns.

As described in the above sections, armoring of channels, revegetation, grade reduction (drop) structures, and other methods will be employed to reduce erosion in the flood routing systems. Cut slopes will be a maximum of 2:1 for long slopes, and 1 1/2:1 with spaced benches for road relocation and other lesser cuts. The emergency spillway will be constructed to release flows past the toe of the dam, and the groin of the dam will be reinforced as necessary.

Fleetwood Creek, now located in a sideslope constructed drainage channel, will be returned to a more natural channel adjacent to the toe of the coarse tailings dump. The channel will be stabilized with natural materials where possible including vegetation, log structures, and other methods to improve geomorphic stability.

The remaining impoundment wetland will improve surface water quality through natural filtration and settling.

5.9 OTHER CLOSURE ACTIVITIES

Other work that will be completed as part of the impoundment closure will be to remove the Rainy Creek diversion pipeline, remove and reclaim roads, regrade portions of the coarse tailings dump, and plant trees on the downstream face of the tailings dam (below the level of the tailings).

The final construction activity for the impoundment will be to demolish the decant tower and plug its outflow piping with a concrete plug.

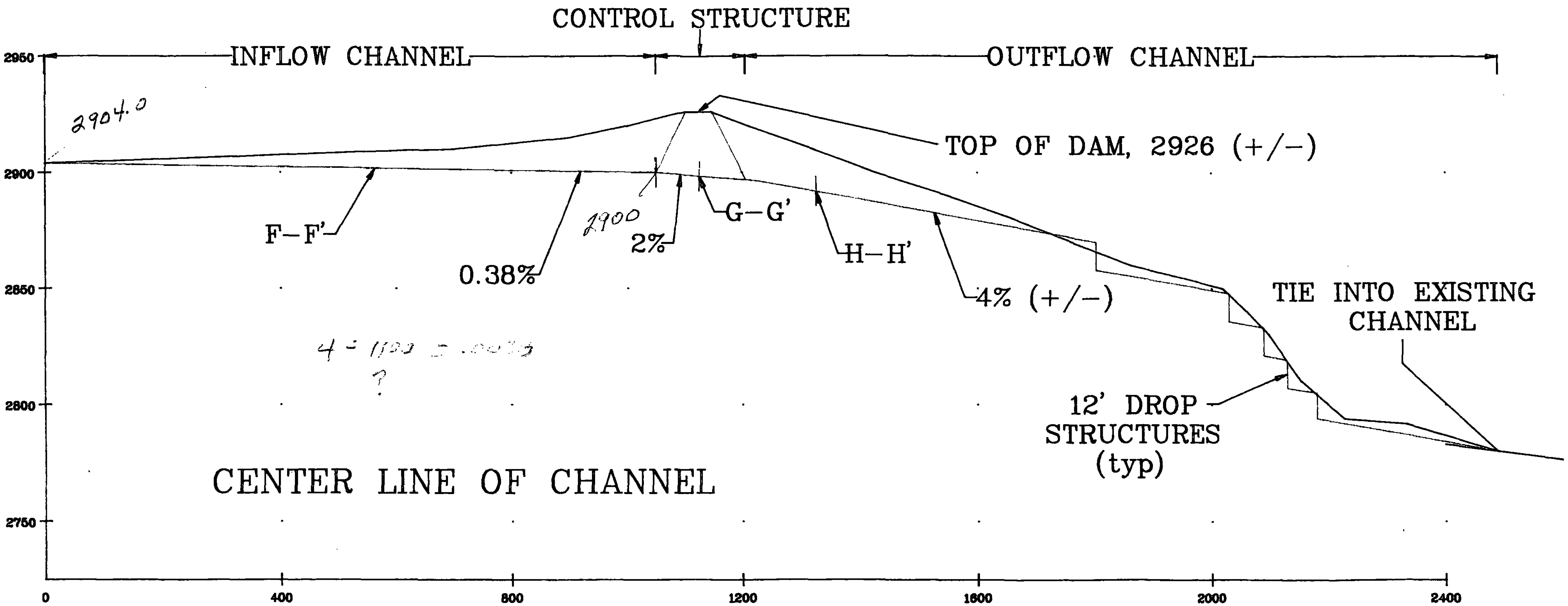

Plate 12. **Section showing centerline of channel for the proposed flood routing system for the vermiculite tailings impoundment.**

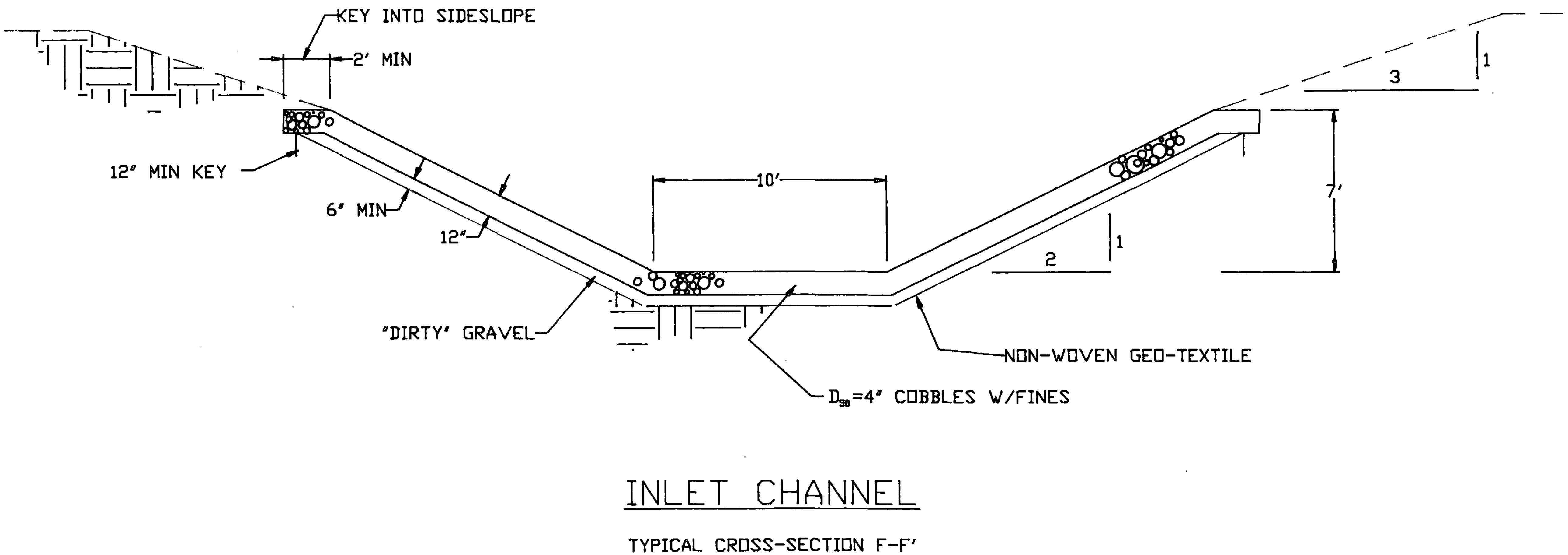

Plate 13. **Typical cross-section of the inflow channel for the proposed flood routing system.**

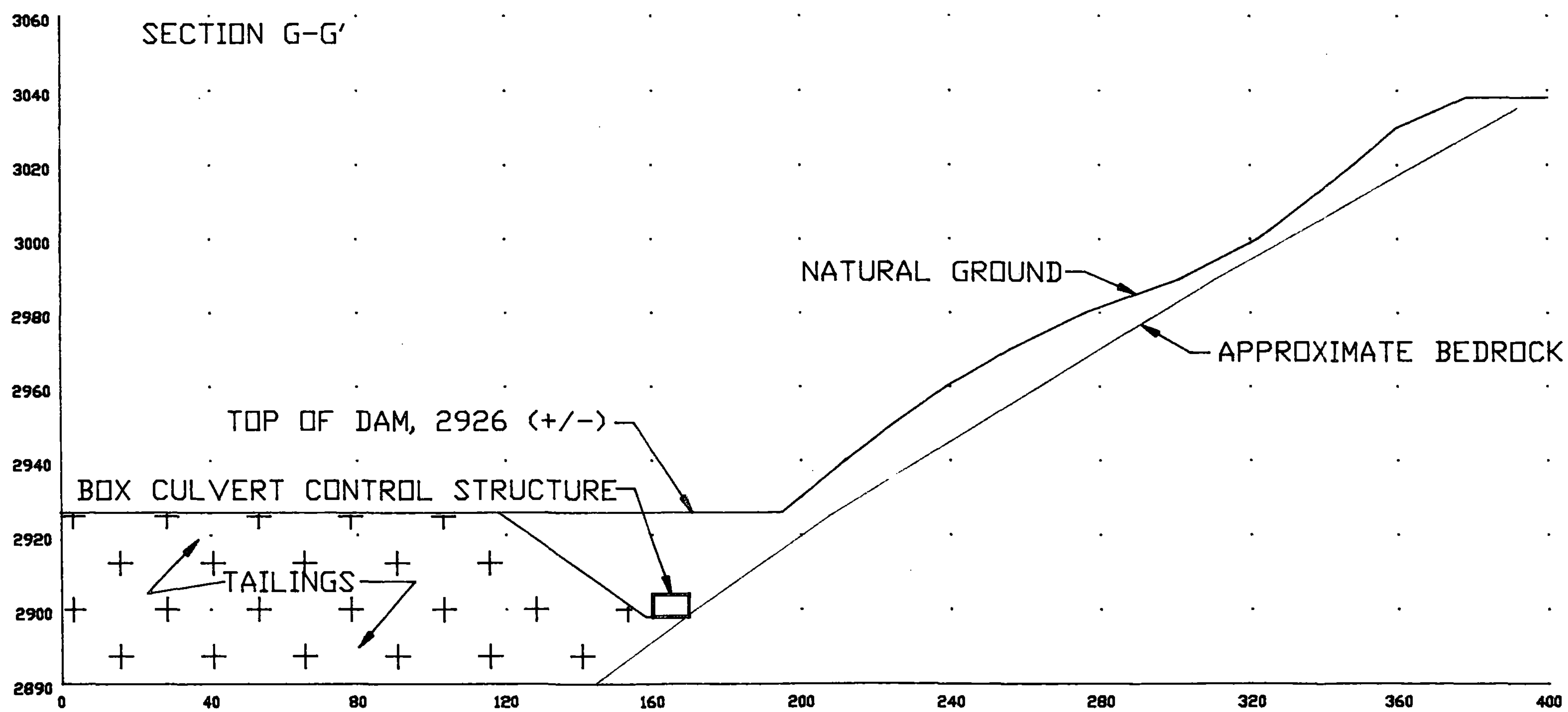

Plate 14. **Typical cross-section of the discharge control structure for the proposed flood routing system. Section taken from centerline of dam.**

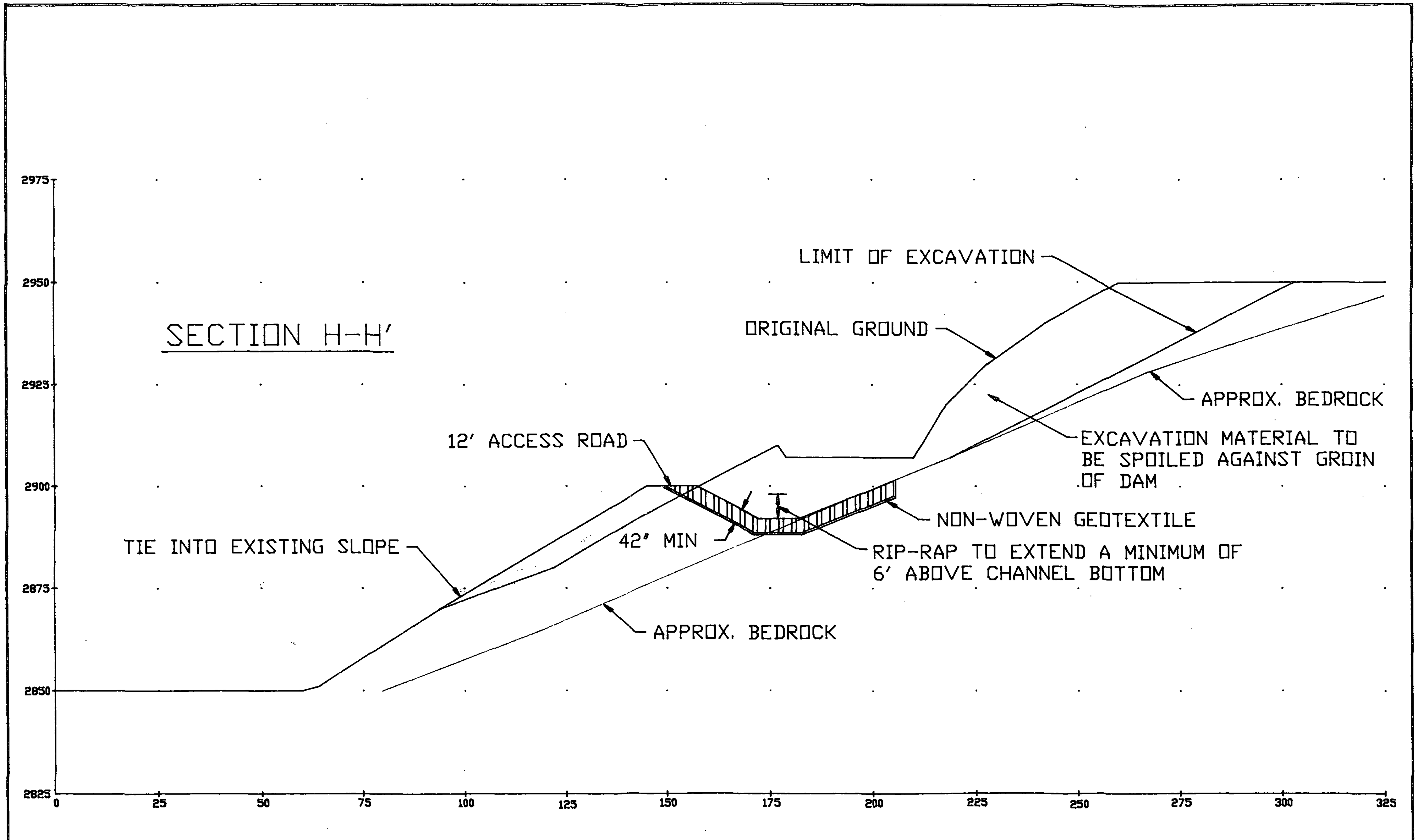

Plate 15. **Typical cross-section of the outflow channel for the proposed flood routing system.**

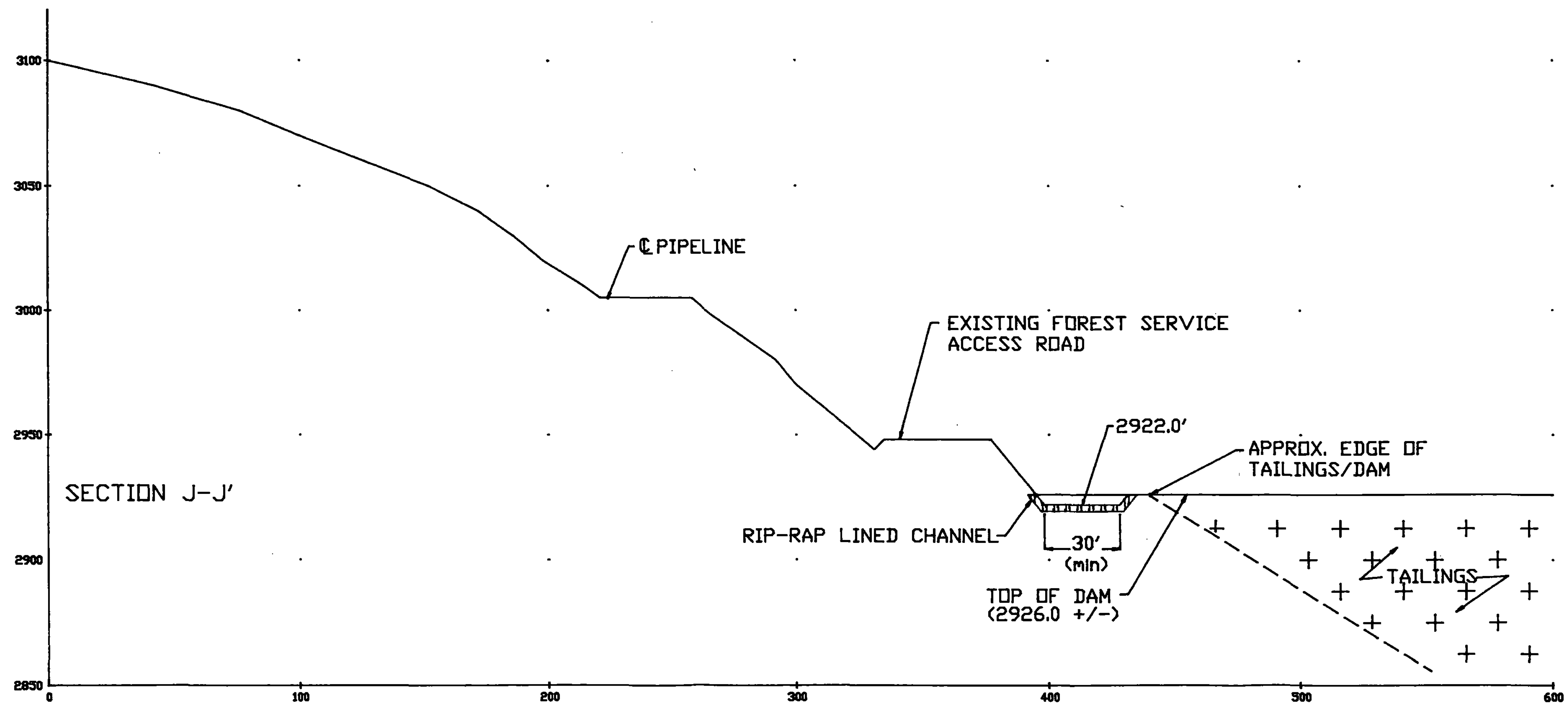

Plate 16. **Typical cross-section of the emergency relief spillway for the vermiculite tailings impoundment.**

6.0 POST-CLOSURE CARE

6.1 POST-CLOSURE MANAGEMENT

W.R. Grace & Company is committed to proper management of the reclaimed mine property as long as it retains ownership of the property. Arrangements would be made for a fulltime custodian to look after the property. Part of the custodian's responsibilities will include periodic inspection of stream routing structures to assure proper operation and structural integrity.

W.R. Grace will close access to the upper mine property. However, situated next to the Forest Service access road, the tailings pond area will be accessible to the public. These areas will be posted for no trespassing. The custodian will provide security for this area to prevent unauthorized access to the property which will assure that initial revegetation efforts are not disturbed by recreational use. The custodian will also be responsible for coordination with regulatory agencies for ongoing monitoring activities.

6.2 WATER QUALITY MONITORING PROGRAM

A program of water quality monitoring was begun in the fall of 1991 by W.R. Grace to develop data regarding current water quality and to monitor the effects of closure activities on future water quality. This program is described in a document submitted to the Montana Department of State Lands, Water Quality Bureau (Hudson, 1991). The program calls for sampling and analysis of Rainy Creek, Fleetwood Creek, Carney Creek and discharges from the tailings impoundment. Monitoring will include heavy metals, although this should not be a problem for this particular mine, and asbestiform fibers. The monitoring will continue for a minimum of three years with provisions for additional monitoring depending on the results of the previous sampling

6.3 MAINTENANCE

The construction of channels for flood routing is not expected to be a solution without maintenance requirements. The recommended alternative is what we believe will offer the lowest maintenance requirements and least potential for catastrophic failures. The success of the closure in meeting these goals for the long-term depends on good maintenance practices. W.R. Grace is committed to this maintenance throughout its ownership of the property and will require that it be continued as a condition of any future sale of the property.

Areas which will require periodic inspection, on at least an annual basis, are the toe drain piping, box culvert outlet structure, and the constructed channels. Should the toe drains begin to fail and remedial action be indicated to prevent saturation and subsequent erosion of the dam foundations, W.R. Grace will implement appropriate corrective measures. A conceptual design for such remedial action has already been prepared by Harding Lawson Associates. Other structures may also require maintenance or reconstruction from time to time to assure continued functionality according to intended design.

7.0 REFERENCES

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APPENDIX A

HYDROLOGIC MODELING RESULTS

W.R. GRACE HYDROGRAPH
FLEETWOOD CREEK
10-YEAR, 24-HOUR STORM (2.4 in.)

	INPUT DATA	TIME STEP (Hrs)	TIME (Hrs)	CUMULATIVE PRECIP (Inches)	CUMULATIVE RUNOFF (Inches)	INCREMENTAL RUNOFF (Inches)	TOTAL FLOW (cfs)
Total Precip (Inches)	2.400	1.000	0.515	0.012	0.000	0.000	0.000
Total Duration (Hrs)	24.000	2.000	1.030	0.026	0.000	0.000	0.000
		3.000	1.545	0.041	0.000	0.000	0.000
Area (Sq. miles)	3.500	4.000	2.061	0.055	0.000	0.000	0.000
Longest Run (Feet)	16370.000	5.000	2.576	0.070	0.000	0.000	0.000
Ave. Slope (%)	11.100	6.000	3.091	0.084	0.000	0.000	0.000
SCS Curve #	60.000	7.000	3.606	0.098	0.000	0.000	0.000
		8.000	4.121	0.115	0.000	0.000	0.000
Storage S (Inches)	6.667	9.000	4.636	0.134	0.000	0.000	0.000
Initial Abstr. (Inches)	1.333	10.000	5.151	0.154	0.000	0.000	0.000
Time-concentration (Hrs)	2.576	11.000	5.667	0.173	0.000	0.000	0.000
Time-peak (Hrs)	1.803	12.000	6.182	0.192	0.000	0.000	0.000
Time-base (Hrs)	4.814	13.000	6.697	0.216	0.000	0.000	0.000
Duration (Hrs)	*0.515*	*14.000*	7.212	0.240	*0.000*	*0.000*	*0.000*
Incr. Precip. (Inches)	0.052	15.000	7.727	0.264	0.000	0.000	0.000
		16.000	8.242	0.288	0.000	0.000	0.000
		17.000	8.757	0.336	0.000	0.000	0.000
		18.000	9.273	0.372	0.000	0.000	0.000
		19.000	9.788	0.413	0.000	0.000	0.000
		20.000	10.303	0.458	0.000	0.000	0.000
		21.000	10.818	0.523	0.000	0.000	0.000
		22.000	11.333	0.617	0.000	0.000	0.000
		23.000	11.848	0.929	0.000	0.019	0.883
		24.000	12.363	1.697	0.019	0.014	4.287
		25.000	12.879	1.819	0.033	0.011	11.824
		26.000	13.394	1.898	0.044	0.009	23.084
		27.000	13.909	1.956	0.053	0.008	34.836
		28.000	14.424	2.002	0.061	0.006	42.210
		29.000	14.939	2.038	0.067	0.006	45.137
		30.000	15.454	2.071	0.074	0.006	44.611
		31.000	15.969	2.100	0.079	0.006	42.476
		32.000	16.485	2.129	0.085	0.005	39.969
		33.000	17.000	2.155	0.090	0.008	37.712
		34.000	17.515	2.191	0.098	0.005	35.928
		35.000	18.030	2.213	0.102	0.004	34.698
		36.000	18.545	2.232	0.107	0.004	33.616
		37.000	19.060	2.251	0.111	0.004	32.267
		38.000	19.575	2.270	0.115	0.004	30.372
		39.000	20.091	2.287	0.119	0.003	28.474
		40.000	20.606	2.302	0.123	0.003	26.701
		41.000	21.121	2.316	0.126	0.003	25.088
		42.000	21.636	2.330	0.130	0.004	23.595
		43.000	22.151	2.345	0.133	0.004	22.301
		44.000	22.666	2.359	0.137	0.004	21.310
		45.000	23.181	2.374	0.140	0.004	20.625
		46.000	23.697	2.388	0.144	0.003	20.132
		47.000	24.212	2.400	0.147	0.000	19.609
		48.000	24.727	2.400	0.147	0.000	18.607
		49.000	25.242	2.400	0.147	0.000	16.698
		50.000	25.757	2.400	0.147	0.000	13.836
		51.000	26.272	2.400	0.147	0.000	10.561
		52.000	26.787	2.400	0.147	0.000	7.712
		53.000	27.303	2.400	0.147	0.000	5.510
		54.000	27.818	2.400	0.147	0.000	3.948
		55.000	28.333	2.400	0.147	0.000	2.842
		56.000	28.848	2.400	0.147	0.000	2.035
		57.000	29.363	2.400	0.147	0.000	1.456
		58.000	29.878	2.400	0.147	0.000	1.040
		59.000	30.393	2.400	0.147	0.000	0.740
		60.000	30.909	2.400	0.147	0.000	0.524
		61.000	31.424	2.400	0.147	0.000	0.370
		62.000	31.939	2.400	0.147	0.000	0.259
		63.000	32.454	2.400	0.147	0.000	0.178
		64.000	32.969	2.400	0.147	0.000	0.120
		65.000	33.484	2.400	0.147	0.000	0.078
		66.000	33.999	2.400	0.147	0.000	0.047
		67.000	34.515	2.400	0.147	0.000	0.025
		68.000	35.030	2.400	0.147	0.000	0.009
		69.000	35.545	2.400	0.147	0.000	0.000
		70.000	36.060	2.400	0.147	0.000	0.000

W.R. GRACE HYDROGRAPH
RAINY CREEK
10-YEAR, 24-HOUR STORM (2.4 In.)

	INPUT DATA	TIME STEP (Hrs)	TIME (Hrs)	CUMULATIVE PRECIP (Inches)	CUMULATIVE RUNOFF (Inches)	INCREMENTAL RUNOFF (Inches)	TOTAL FLOW (cfs)
Total Precip (Inches)	2.400	1.000	0.709	0.012	0.000	0.000	0.000
Total Duration (Hrs)	24.000	2.000	1.417	0.034	0.000	0.000	0.000
		3.000	2.126	0.055	0.000	0.000	0.000
Area (Sq. miles)	5.900	4.000	2.834	0.077	0.000	0.000	0.000
Longest Run (Feet)	25870.000	5.000	3.543	0.098	0.000	0.000	0.000
Ave. Slope (%)	12.200	6.000	4.252	0.125	0.000	0.000	0.000
SCS Curve #	60.000	7.000	4.960	0.144	0.000	0.000	0.000
		8.000	5.669	0.173	0.000	0.000	0.000
Storage S (Inches)	6.667	9.000	6.377	0.204	0.000	0.000	0.000
Initial Abstr. (Inches)	1.333	10.000	7.086	0.240	0.000	0.000	0.000
Time-concentration (Hrs)	3.543	11.000	7.795	0.276	0.000	0.000	0.000
Time-peak (Hrs)	2.480	12.000	8.503	0.319	0.000	0.000	0.000
Time-base (Hrs)	6.622	13.000	9.212	0.353	0.000	0.000	0.000
Duration (Hrs)	0.709	14.000	9.921	0.413	0.000	0.000	0.000
Incr. Precip. (Inches)	0.071	15.000	10.629	0.487	0.000	0.000	0.000
		16.000	11.338	0.617	0.000	0.010	0.553
		17.000	12.046	1.591	0.010	0.023	3.614
		18.000	12.755	1.819	0.033	0.011	11.528
		19.000	13.464	1.898	0.044	0.013	25.239
		20.000	14.172	1.980	0.057	0.010	41.973
		21.000	14.881	2.038	0.067	0.009	55.838
		22.000	15.589	2.086	0.076	0.009	63.014
		23.000	16.298	2.129	0.085	0.008	65.494
		24.000	17.007	2.167	0.093	0.005	64.305
		25.000	17.715	2.191	0.098	0.007	61.614
		26.000	18.424	2.222	0.105	0.006	58.121
		27.000	19.132	2.251	0.111	0.007	54.338
		28.000	19.841	2.280	0.118	0.005	50.903
		29.000	20.550	2.302	0.123	0.005	48.260
		30.000	21.258	2.323	0.128	0.004	45.735
		31.000	21.967	2.338	0.131	0.005	43.092
		32.000	22.676	2.359	0.137	0.005	40.268
		33.000	23.384	2.381	0.142	0.005	37.818
		34.000	24.093	2.400	0.147	0.000	35.807
		35.000	24.801	2.400	0.147	0.000	33.788
		36.000	25.510	2.400	0.147	0.000	30.448
		37.000	26.219	2.400	0.147	0.000	25.435
		38.000	26.927	2.400	0.147	0.000	19.506
		39.000	27.636	2.400	0.147	0.000	14.248
		40.000	28.344	2.400	0.147	0.000	10.133
		41.000	29.053	2.400	0.147	0.000	7.251
		42.000	29.762	2.400	0.147	0.000	5.209
		43.000	30.470	2.400	0.147	0.000	3.733
		44.000	31.179	2.400	0.147	0.000	2.664
		45.000	31.887	2.400	0.147	0.000	1.897
		46.000	32.596	2.400	0.147	0.000	1.346
		47.000	33.305	2.400	0.147	0.000	0.957
		48.000	34.013	2.400	0.147	0.000	0.672
		49.000	34.722	2.400	0.147	0.000	0.465
		50.000	35.430	2.400	0.147	0.000	0.317
		51.000	36.139	2.400	0.147	0.000	0.214
		52.000	36.848	2.400	0.147	0.000	0.140
		53.000	37.556	2.400	0.147	0.000	0.089
		54.000	38.265	2.400	0.147	0.000	0.047
		55.000	38.974	2.400	0.147	0.000	0.017
		56.000	39.682	2.400	0.147	0.000	0.000
		57.000	40.391	2.400	0.147	0.000	0.000
		58.000	41.099	2.400	0.147	0.000	0.000
		59.000	41.808	2.400	0.147	0.000	0.000
		60.000	42.517	2.400	0.147	0.000	0.000
		61.000	43.225	2.400	0.147	0.000	0.000
		62.000	43.934	2.400	0.147	0.000	0.000
		63.000	44.642	2.400	0.147	0.000	0.000
		64.000	45.351	2.400	0.147	0.000	0.000
		65.000	46.060	2.400	0.147	0.000	0.000
		66.000	46.768	2.400	0.147	0.000	0.000
		67.000	47.477	2.400	0.147	0.000	0.000
		68.000	48.185	2.400	0.147	0.000	0.000
		69.000	48.894	2.400	0.147	0.000	0.000
		70.000	49.603	2.400	0.147	0.000	0.000

W.R. GRACE HYDROGRAPH
RAINY CREEK
100-YEAR, 24-HOUR STORM (3.4 In.)

	INPUT DATA	TIME STEP (Hrs)	TIME (Hrs)	CUMULATIVE PRECIP (Inches)	CUMULATIVE RUNOFF (Inches)	INCREMENTAL RUNOFF (Inches)	TOTAL FLOW (cfs)
Total Precip (Inches)	3.400	1.000	0.709	0.017	0.000	0.000	0.000
Total Duration (Hrs)	24.000	2.000	1.417	0.048	0.000	0.000	0.000
		3.000	2.126	0.078	0.000	0.000	0.000
Area (Sq. miles)	5.900	4.000	2.834	0.109	0.000	0.000	0.000
Longest Run (Feet)	25870.000	5.000	3.543	0.139	0.000	0.000	0.000
Ave. Slope (%)	12.200	6.000	4.252	0.177	0.000	0.000	0.000
SCS Curve #	60.000	7.000	4.960	0.204	0.000	0.000	0.000
		8.000	5.669	0.245	0.000	0.000	0.000
Storage S (Inches)	6.667	9.000	6.377	0.289	0.000	0.000	0.000
Initial Abstr. (Inches)	1.333	10.000	7.086	0.340	0.000	0.000	0.000
Time-concentration (Hrs)	3.543	11.000	7.795	0.391	0.000	0.000	0.000
Time-peak (Hrs)	2.460	12.000	8.503	0.452	0.000	0.000	0.000
Time-base (Hrs)	6.622	13.000	9.212	0.500	0.000	0.000	0.000
Duration (Hrs)	0.709	14.000	9.921	0.585	0.000	0.000	0.000
Incr. Precip. (Inches)	0.100	15.000	10.629	0.690	0.000	0.000	0.000
		16.000	11.338	0.874	0.000	0.112	6.434
		17.000	12.046	2.254	0.112	0.084	31.205
		18.000	12.755	2.577	0.196	0.034	84.132
		19.000	13.464	2.689	0.229	0.037	159.174
		20.000	14.172	2.805	0.266	0.027	229.974
		21.000	14.881	2.887	0.294	0.024	262.377
		22.000	15.589	2.955	0.317	0.022	262.018
		23.000	16.298	3.016	0.339	0.020	244.487
		24.000	17.007	3.070	0.359	0.013	220.958
		25.000	17.715	3.104	0.372	0.017	198.889
		26.000	18.424	3.148	0.388	0.016	178.352
		27.000	19.132	3.189	0.404	0.016	159.606
		28.000	19.841	3.230	0.420	0.012	143.976
		29.000	20.550	3.261	0.432	0.012	131.924
		30.000	21.258	3.291	0.444	0.008	121.475
		31.000	21.967	3.312	0.453	0.012	111.794
		32.000	22.676	3.342	0.465	0.013	102.432
		33.000	23.384	3.373	0.478	0.011	94.523
		34.000	24.093	3.400	0.489	0.000	88.118
		35.000	24.801	3.400	0.489	0.000	82.102
		36.000	25.510	3.400	0.489	0.000	73.335
		37.000	26.219	3.400	0.489	0.000	60.936
		38.000	26.927	3.400	0.489	0.000	46.571
		39.000	27.636	3.400	0.489	0.000	33.759
		40.000	28.344	3.400	0.489	0.000	23.939
		41.000	29.053	3.400	0.489	0.000	17.109
		42.000	29.762	3.400	0.489	0.000	12.273
		43.000	30.470	3.400	0.489	0.000	8.786
		44.000	31.179	3.400	0.489	0.000	6.262
		45.000	31.887	3.400	0.489	0.000	4.455
		46.000	32.596	3.400	0.489	0.000	3.157
		47.000	33.305	3.400	0.489	0.000	2.241
		48.000	34.013	3.400	0.489	0.000	1.571
		49.000	34.722	3.400	0.489	0.000	1.087
		50.000	35.430	3.400	0.489	0.000	0.738
		51.000	36.139	3.400	0.489	0.000	0.497
		52.000	36.848	3.400	0.489	0.000	0.325
		53.000	37.556	3.400	0.489	0.000	0.207
		54.000	38.265	3.400	0.489	0.000	0.109
		55.000	38.974	3.400	0.489	0.000	0.039
		56.000	39.682	3.400	0.489	0.000	0.000
		57.000	40.391	3.400	0.489	0.000	0.000
		58.000	41.099	3.400	0.489	0.000	0.000
		59.000	41.808	3.400	0.489	0.000	0.000
		60.000	42.517	3.400	0.489	0.000	0.000
		61.000	43.225	3.400	0.489	0.000	0.000
		62.000	43.934	3.400	0.489	0.000	0.000
		63.000	44.642	3.400	0.489	0.000	0.000
		64.000	45.351	3.400	0.489	0.000	0.000
		65.000	46.060	3.400	0.489	0.000	0.000
		66.000	46.768	3.400	0.489	0.000	0.000
		67.000	47.477	3.400	0.489	0.000	0.000
		68.000	48.185	3.400	0.489	0.000	0.000
		69.000	48.894	3.400	0.489	0.000	0.000
		70.000	49.603	3.400	0.489	0.000	0.000

W.R. GRACE HYDROGRAPH
FLEETWOOD CREEK
100-YEAR, 24-HOUR STORM (3.4 In.)

	INPUT DATA	TIME STEP (Hrs)	TIME (Hrs)	CUMULATIVE PRECIP (Inches)	CUMULATIVE RUNOFF (Inches)	INCREMENTAL RUNOFF (Inches)	TOTAL FLOW (cfs)
Total Precip (Inches)	3.400	1.000	0.515	0.017	0.000	0.000	0.000
Total Duration (Hrs)	24.000	2.000	1.030	0.037	0.000	0.000	0.000
		3.000	1.545	0.058	0.000	0.000	0.000
Area (Sq. miles)	3.500	4.000	2.061	0.078	0.000	0.000	0.000
Longest Run (Feet)	16370.000	5.000	2.576	0.099	0.000	0.000	0.000
Ave. Slope (%)	11.100	6.000	3.091	0.119	0.000	0.000	0.000
SCS Curve #	60.000	7.000	3.606	0.139	0.000	0.000	0.000
		8.000	4.121	0.163	0.000	0.000	0.000
Storage S (Inches)	6.667	9.000	4.636	0.190	0.000	0.000	0.000
Initial Abstr. (Inches)	1.333	10.000	5.151	0.218	0.000	0.000	0.000
Time-concentration (Hrs)	2.576	11.000	5.667	0.245	0.000	0.000	0.000
Time-peak (Hrs)	1.803	12.000	6.182	0.272	0.000	0.000	0.000
Time-base (Hrs)	4.814	13.000	6.697	0.306	0.000	0.000	0.000
Duration (Hrs)	0.515	14.000	7.212	0.340	0.000	0.000	0.000
Incr. Precip. (Inches)	0.073	15.000	7.727	0.374	0.000	0.000	0.000
		16.000	8.242	0.408	0.000	0.000	0.000
		17.000	8.757	0.476	0.000	0.000	0.000
		18.000	9.273	0.527	0.000	0.000	0.000
		19.000	9.788	0.585	0.000	0.000	0.000
		20.000	10.303	0.649	0.000	0.000	0.000
		21.000	10.818	0.741	0.000	0.000	0.000
		22.000	11.333	0.874	0.000	0.000	0.000
		23.000	11.848	1.316	0.000	0.148	6.958
		24.000	12.363	2.404	0.148	0.047	30.756
		25.000	12.879	2.577	0.196	0.034	78.213
		26.000	13.394	2.689	0.229	0.026	139.952
		27.000	13.909	2.771	0.255	0.021	191.041
		28.000	14.424	2.836	0.276	0.017	203.807
		29.000	14.939	2.887	0.294	0.016	194.919
		30.000	15.454	2.934	0.310	0.014	173.976
		31.000	15.969	2.975	0.324	0.015	152.814
		32.000	16.485	3.016	0.339	0.014	135.109
		33.000	17.000	3.053	0.353	0.019	120.709
		34.000	17.515	3.104	0.372	0.012	109.377
		35.000	18.030	3.135	0.383	0.010	101.135
		36.000	18.545	3.162	0.394	0.011	94.390
		37.000	19.060	3.189	0.404	0.011	87.925
		38.000	19.575	3.216	0.415	0.009	80.838
		39.000	20.091	3.240	0.424	0.008	74.278
		40.000	20.606	3.261	0.432	0.008	68.438
		41.000	21.121	3.281	0.440	0.008	63.336
		42.000	21.636	3.301	0.449	0.008	58.801
		43.000	22.151	3.322	0.457	0.008	54.947
		44.000	22.666	3.342	0.465	0.008	51.950
		45.000	23.181	3.363	0.474	0.008	49.740
		46.000	23.697	3.383	0.482	0.007	47.935
		47.000	24.212	3.400	0.489	0.000	46.378
		48.000	24.727	3.400	0.489	0.000	43.786
		49.000	25.242	3.400	0.489	0.000	39.153
		50.000	25.757	3.400	0.489	0.000	32.371
		51.000	26.272	3.400	0.489	0.000	24.684
		52.000	26.787	3.400	0.489	0.000	18.018
		53.000	27.303	3.400	0.489	0.000	12.869
		54.000	27.818	3.400	0.489	0.000	9.219
		55.000	28.333	3.400	0.489	0.000	6.633
		56.000	28.848	3.400	0.489	0.000	4.747
		57.000	29.363	3.400	0.489	0.000	3.394
		58.000	29.878	3.400	0.489	0.000	2.425
		59.000	30.393	3.400	0.489	0.000	1.723
		60.000	30.909	3.400	0.489	0.000	1.219
		61.000	31.424	3.400	0.489	0.000	0.860
		62.000	31.939	3.400	0.489	0.000	0.601
		63.000	32.454	3.400	0.489	0.000	0.414
		64.000	32.969	3.400	0.489	0.000	0.279
		65.000	33.484	3.400	0.489	0.000	0.181
		66.000	33.999	3.400	0.489	0.000	0.110
		67.000	34.515	3.400	0.489	0.000	0.057
		68.000	35.030	3.400	0.489	0.000	0.020
		69.000	35.545	3.400	0.489	0.000	0.000
		70.000	36.060	3.400	0.489	0.000	0.000

W.R. GRACE HYDROGRAPH
TAILINGS IMPOUNDMENT WATERSHED
PMF STORM EVENT, 6-HOUR AUGUST THUNDERSTORM (10.7 in.)

Time (Hrs)	Incremental Rainfall (Inches)	Rainfall Rate (In/Hr)	Rainy Creek Flow (cfs)	Fleetwood Creek Flow (cfs)	Combined Flow (cfs)
0.000	0.000	0.000	0.000	0.000	0.000
0.500	0.200	0.400	0.000	0.000	0.000
1.000	0.300	0.600	5.000	6.000	11.000
1.500	0.500	1.000	22.000	35.000	57.000
2.000	2.200	4.400	73.000	165.000	238.000
2.500	4.500	9.000	280.000	447.000	727.000
3.000	1.000	2.000	1131.000	1354.000	2485.000
3.500	0.500	1.000	1782.000	3112.000	4894.000
4.000	0.500	1.000	3242.000	5278.000	8520.000
4.500	0.300	0.600	5582.000	5884.000	11466.000
5.000	0.300	0.600	6900.000	4776.000	11676.000
5.500	0.200	0.400	7330.000	3652.000	10982.000
6.000	0.200	0.400	6350.000	2917.000	9267.000
6.500	0.000	0.000	5012.000	2380.000	7392.000
7.000	0.000	0.000	4256.000	1956.000	6212.000
7.500	0.000	0.000	3560.000	1604.000	5164.000
8.000	0.000	0.000	2960.000	1350.000	4310.000
8.500	0.000	0.000	2606.000	1123.000	3729.000
9.000	0.000	0.000	2218.000	940.000	3158.000
9.500	0.000	0.000	1921.000	800.000	2721.000
10.000	0.000	0.000	1675.000	680.000	2355.000
10.500	0.000	0.000	1466.000	578.000	2044.000
11.000	0.000	0.000	1291.000	510.000	1801.000
11.500	0.000	0.000	1197.000	439.000	1636.000
12.000	0.000	0.000	1076.000	377.000	1453.000
12.500	0.000	0.000	963.000	322.000	1285.000
13.000	0.000	0.000	890.000	270.000	1160.000
13.500	0.000	0.000	804.000	213.000	1017.000
14.000	0.000	0.000	722.000	169.000	891.000
14.500	0.000	0.000	648.000	117.000	765.000
15.000	0.000	0.000	604.000	72.000	676.000
15.500	0.000	0.000	556.000	31.000	587.000
16.000	0.000	0.000	505.000	18.000	523.000
16.500	0.000	0.000	459.000	11.000	470.000
17.000	0.000	0.000	420.000	6.000	426.000
17.500	0.000	0.000	375.000	3.000	378.000
18.000	0.000	0.000	336.000		336.000
18.500	0.000	0.000	314.000		314.000
19.000	0.000	0.000	280.000		280.000
19.500	0.000	0.000	243.000		243.000
20.000	0.000	0.000	204.000		204.000
20.500	0.000	0.000	164.000		164.000
21.000	0.000	0.000	115.000		115.000
21.500	0.000	0.000	43.000		43.000
22.000	0.000	0.000	25.000		25.000
22.500	0.000	0.000	17.000		17.000
23.000	0.000	0.000	9.000		9.000
23.500	0.000	0.000	5.000		5.000
24.000	0.000	0.000	2.000		2.000

APPENDIX B

PROBABLE MAXIMUM FLOOD CALCULATIONS

→ USE METHOD FOR THUNDERSTORM PMP – pg 180 - 184 OF HMR 43 ←

1) DISTANCE FROM S.E. BORDER : ~ 435 miles

2) MAY : 71.5 %
JUNE : 79.1 %
JULY : 84.4 %
AUG : 85.6 % ← USE
SEPT : 74.9 %

NOTE: THIS METHOD IS TAKEN DIRECTLY FROM HMR 43 : "PROBABLE MAXIMUM PRECIPITATION, NORTHWEST STATES", U.S. WEATHER BUREAU, 1966

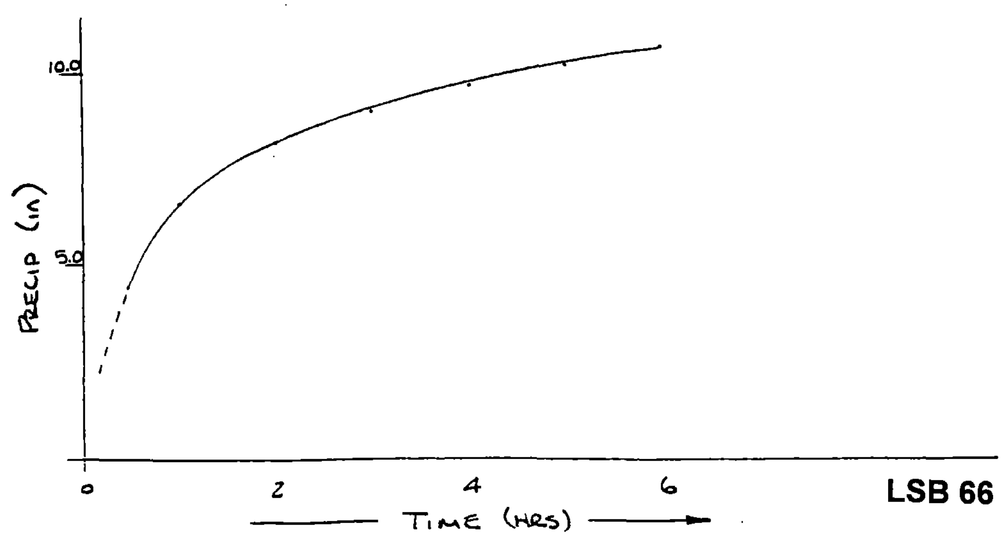
3) 2900' ELEVATION → ≤ 5000'

4) ½ HR : 6.0 in
1 HR : 8.0 in
2 HR 9.5 in
3 HR : 10.3 in
4 HR 10.8 in
5 HR : 11.25 in
6 HR : 11.6 in

5) 1 HR : 84 %
3 HR : 87 %
6 HR : 92 %

6) 1 HR : (8.0) (0.84) = 6.7"
3 HR : (10.3) (0.87) = 9.0"
6 HR : (11.6) (0.92) = 10.7"

7) ½ HR : 4.5"
1 HR : 6.7"
2 HR 8.2"
3 HR : 9.0"
4 HR : 9.7"
5 HR : 10.2"
6 HR : 10.7"



LSB 66 00089

7) CONT.

½	1	2	3	4	5	6
4.5	6.7	8.2	9.0	9.7	10.2	10.7

6.7 1.5 .8 .7 .5 .5

EXPAND FOR ½ HR DURATION: (USING RATIO 4.5/6.7 = 67%)

① ② ③ ④ ⑤ ⑥

0 ½ 1 1½ 2 2½ 3 3½ 4 4½ 5 5½ 6

0 4.5 6.7 7.7 8.2 8.7 9.2 9.5 9.7 10.0 10.2 10.5 10.7

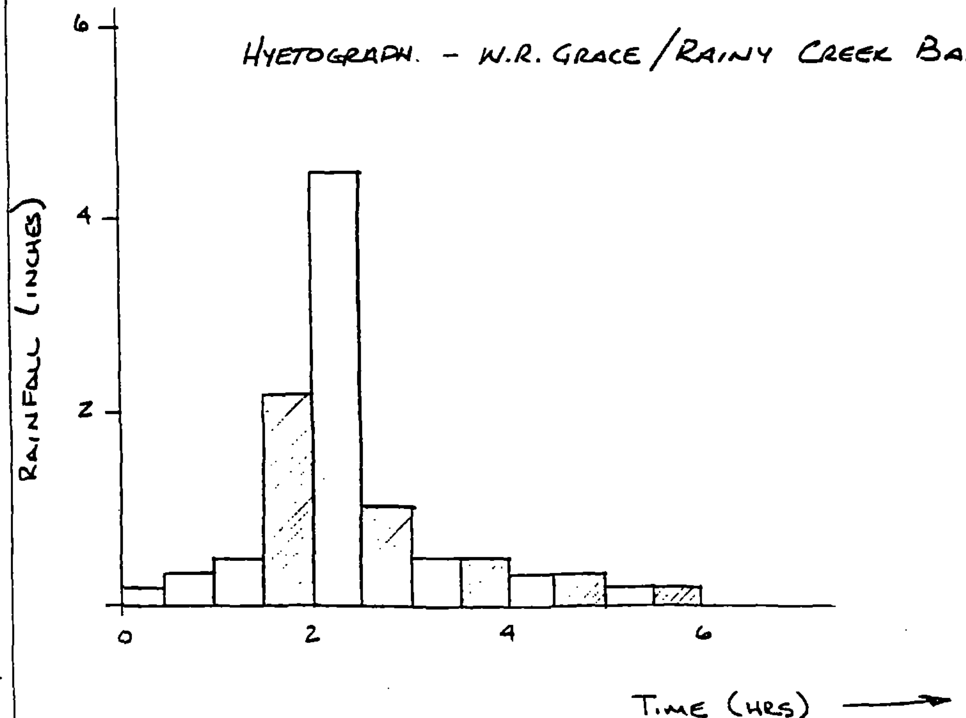
4.5 2.2 1.0 0.5 0.5 0.5 0.3 0.2 0.3 0.2 0.3 0.2

1 2 3 4 5 6 7 10 8 11 9 12

8) SEQUENCE (½ HR DURATION)

0.2 0.3 0.5 2.2 4.5 1.0 0.5 0.5 0.3 0.3 0.2 0.2

NOTE: THESE VALUES WILL BE USED TO CALCULATE RUNOFF BY BUREAU OF RECLAMATION METHOD OUTLINED IN CHPT 4 OF "FLOOD HYDROLOGY MANUAL", BUREAU OF RECLAMATION, 1989.



→ CALCULATE RUNOFF FOR RAINY CREEK: ← *

RAINY CREEK: 5.9 SQ MI
25872 FT (4.9 MI) LONGEST REACH
12.2% (637'/MI) SLOPE

$$\text{Lag} = L_g = 26\, K_n \left(\frac{L L_{ca}}{S^{0.5}}\right)^{0.33}$$

where $L = 4.9$ mi
$L_{ca} = \sim 2.8$ mi
$S = 637'/\text{mi}$
$K_n = 0.17$ (estimated from Table 4.5)

$$\therefore L_g = 26(0.17)\left(\frac{(4.9)(2.8)}{\sqrt{637}}\right)^{0.33}$$

$$= 3.61 \text{ HRS}$$

DURATION $\sim L_g/5.5 = 3.61/5.5 = 0.65$ HR $= \sim 39$ minutes

CHECKING 15 to 20% OF LAG $0.15(3.61)60 = 32.5$ minutes ✓

→ USE 30 MINUTE DURATION ←

INFILTRATION LOSSES: SOIL TYPE 'B' · 0.15 TO 0.30 "/HR ULT. INFILTRATIO.

PMP METHOD REQUIRES SATURATED SOILS W/ ULT. INFILTRATION → THIS AREA IS FOREST W/ EXC. GROUND COVER: USE 0.25"/HR

CALCULATE "S-graph"

$$q_{ult} \text{ (ultimate discharge)} = \frac{645.3 \times 5.9}{0.5} = 7615 \text{ hrs-cfs}$$

— NOTE: USE TABLE 4-16 (CASCADE AREA) FOR S-graph data.

LSB 66 00091

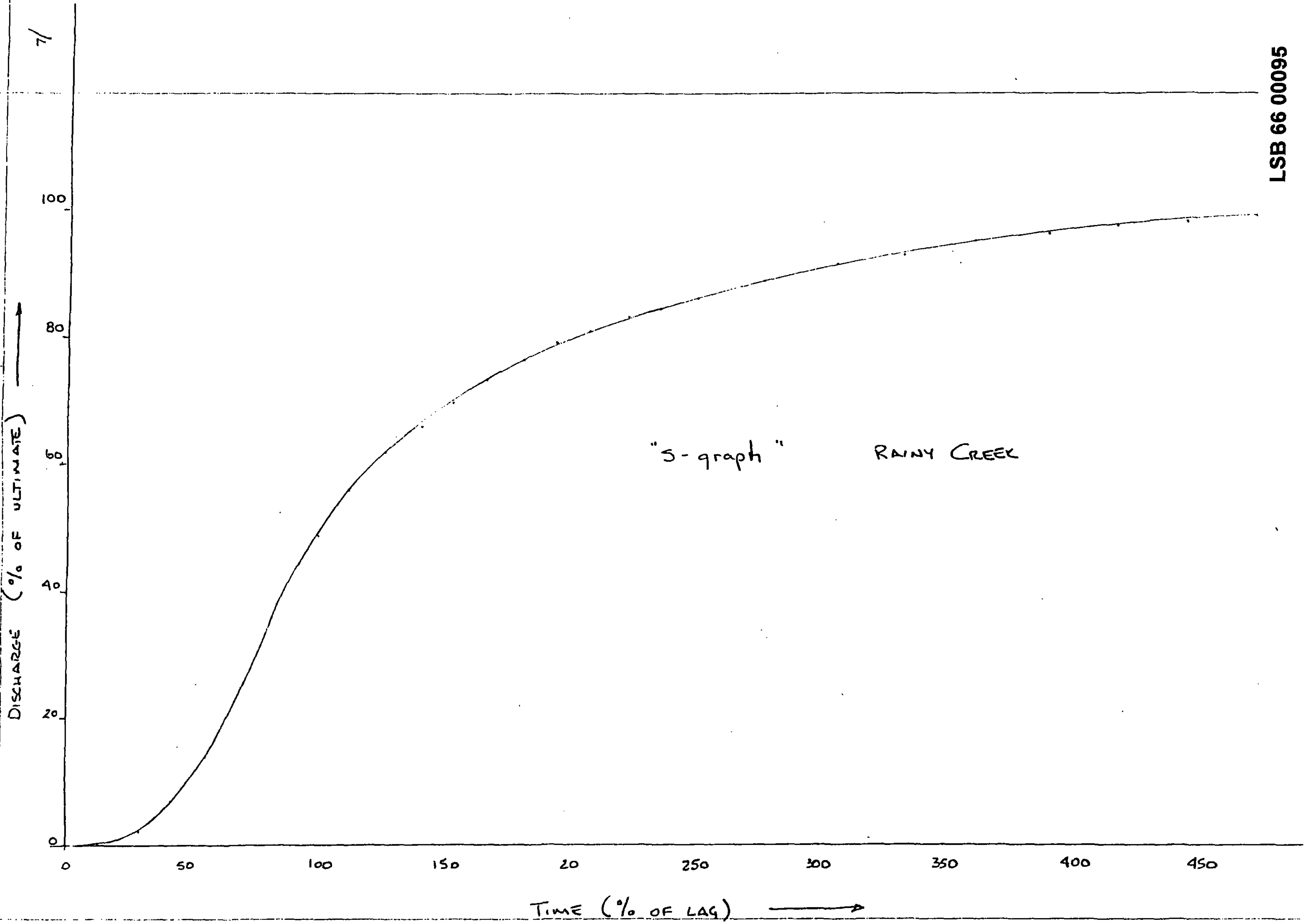
* NOTE: THIS METHOD IS TAKEN FROM CHPT 4, "FLOOD HYDROLOGY MANUAL", BUREAU OF REC. 1989.

TIME HRS	ACCUM RAINFALL	ACCUM RUNOFF	INCREM RUNOFF	REVERSED INCREM. RUNOFF
0	0	0	0	.07
0.5	0.2	.08	.08	.08
1.0	0.5	.25	.17	.17
1.5	1.0	.63	.38	.18
2.0	3.2	2.70	2.07	.37
2.5	7.7	7.08	4.38	.38
3.0	8.7	7.95	.87	.87
3.5	9.2	8.33	.38	4.38
4.0	9.7	8.70	.37	2.07
4.5	10.0	8.88	.18	.38
5.0	10.3	9.05	.17	.17
5.5	10.5	9.13	.08	.08
6.0	10.7	9.20	.07	0

RAINY CREEK

TIME HRS	TIME (% Lg)	DISCHARGE (% ULT.)	Σ HYDROGRAPH ORDINATES (cfs)	UNIT HYDROGRAPH (cfs)	FLOOD HYDROGRAPH (cfs)
0	0	0	0	0	0
0.5	14	0.8	61	61	0
1.0	28	2.7	206	145	5
1.5	42	6.9	525	319	22
2.0	55	14.1	1074	549	73
2.5	69	25.5	1942	868	280
3.0	83	38.2	2909	967	1131
3.5	97	48.8	3716	807	1782
4.0	111	55.8	4249	533	3242
4.5	125	61.6	4691	442	5582
5.0	139	66.3	5049	358	6900
5.5	152	69.9	5323	274	7330
6.0	166	73.3	5582	259	6350
6.5	180	76.15	5799	217	5012
7.0	194	78.7	5993	194	4256
7.5	208	80.95	6164	171	3560
8.0	222	82.95	6317	153	2960
8.5	235	84.63	6445	128	2606
9.0	249	86.3	6572	127	2218
9.5	263	87.8	6686	114	1921
10.0	277	89.1	6785	99	1675
10.5	291	90.35	6880	95	1466
11.0	305	91.47	6965	85	1291

11.5	319	92.48	7042	77	1197
12.0	332	93.3	7108	66	1076
12.5	346	94.2	7172	64	963
13.0	360	94.96	7231	59	890
13.5	374	95.66	7285	54	804
14.0	388	96.3	7333	48	722
14.5	402	96.9	7378	45	648
15.0	416	97.4	7418	40	604
15.5	429	97.9	7452	34	556
16.0	443	98.3	7486	34	505
16.5	457	98.7	7516	30	459
17.0	471	99.0	7542	26	420
17.5	485	99.33	7564	22	375
18.0	498	99.56	7581	17	336
18.5	512	99.75	7596	15	314
19.0					280
19.5					243
20.0					204
					164
					115
					43
					25
					17
					9
					5


"S-graph"
RAINY CREEK
DISCHARGE (% OF ULTIMATE)
100
80
60
40
20
0
0
50
100
150
20
250
300
350
400
450
TIME (% OF LAG)

→ RUNOFF FOR FLEETWOOD CREEK ←

FLEETWOOD CREEK: AREA = 3.5 mi^2

L = 3.1 mi

L_{ca} = 1.2 mi

S = 5104 - 2900/3.1 = 711'/mi

$$Lag = 26 K_n \left(\frac{L L_{ca}}{S^{0.5}} \right)^{0.33}$$

$$= 26 (0.17) \left(\frac{(3.1)(1.2)}{\sqrt{711}} \right)^{0.33} = 2.29 \text{ HRS}$$

DURATION = $L_g / 5.5$ = $(2.29)(60)/5.5$ = 25 min → USE 30 min (TO BE CONS. w/ RAINY)

INFILTRATION: 0.25"/HR

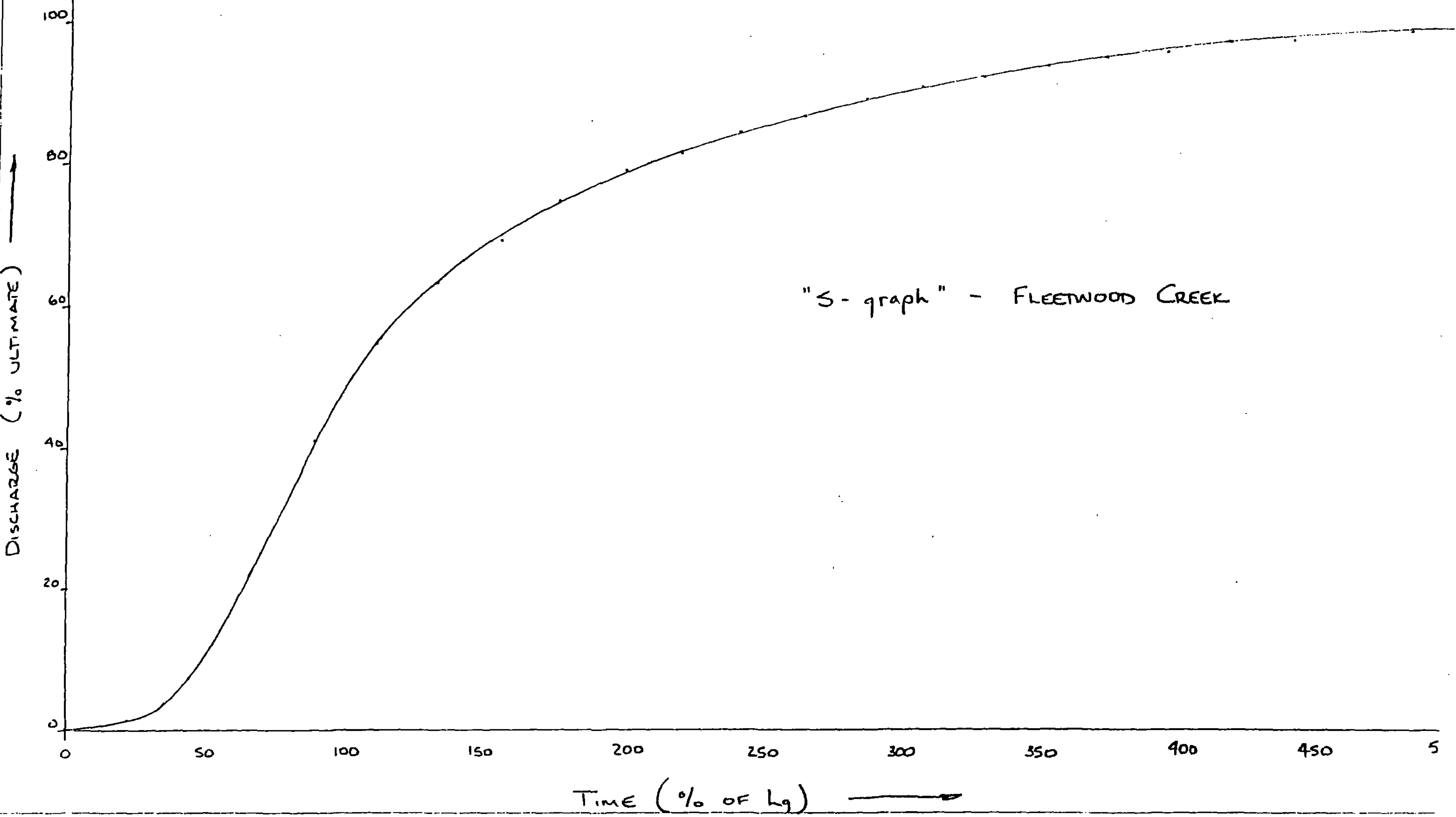
$$q_{ULT} = \frac{645.3 \times 3.5}{0.5} = 4517 \text{ hrs-cfs}$$

→ USE TABLE 4-16 (CASCADES)

FLEETWOOD CREEK

TIME HRS	TIME (%Lg)	DISCHARGE (% ULT)	Σ HYDROGRAPH ORDINATES (cfs)	UNIT HYDROGRAPH (cfs)	FLOOD HYDROGRAPH (cfs)
0	0	0	0	0	0
0.5	22	1.7	78	78	0
1.0	44	7.8	351	273	6
1.5	66	22.7	1024	673	35
2.0	87	41.3	1864	840	165
2.5	109	54.8	2476	612	447
3.0	131	63.7	2877	401	1354
3.5	153	70.2	3169	292	3112
4.0	175	75.2	3395	226	5278
4.5	197	79.2	3578	183	5884
5.0	218	82.4	3722	144	4776
5.5	240	85.2	3850	128	3652
6.0	262	87.7	3960	110	2917
6.5	284	89.8	4054	94	2380
7.0	306	91.5	4135	81	1956
7.5	328	93.1	4205	70	1604
8.0	349	94.35	4262	57	1350
8.5	371	95.5	4315	53	1123
9.0	393	96.5	4360	45	940
9.5	415	97.4	4399	39	800
10 0	437	98.1	4432	33	680
10.5	459	98.8	4461	29	578
11.0	480	99.2	4482	21	510.

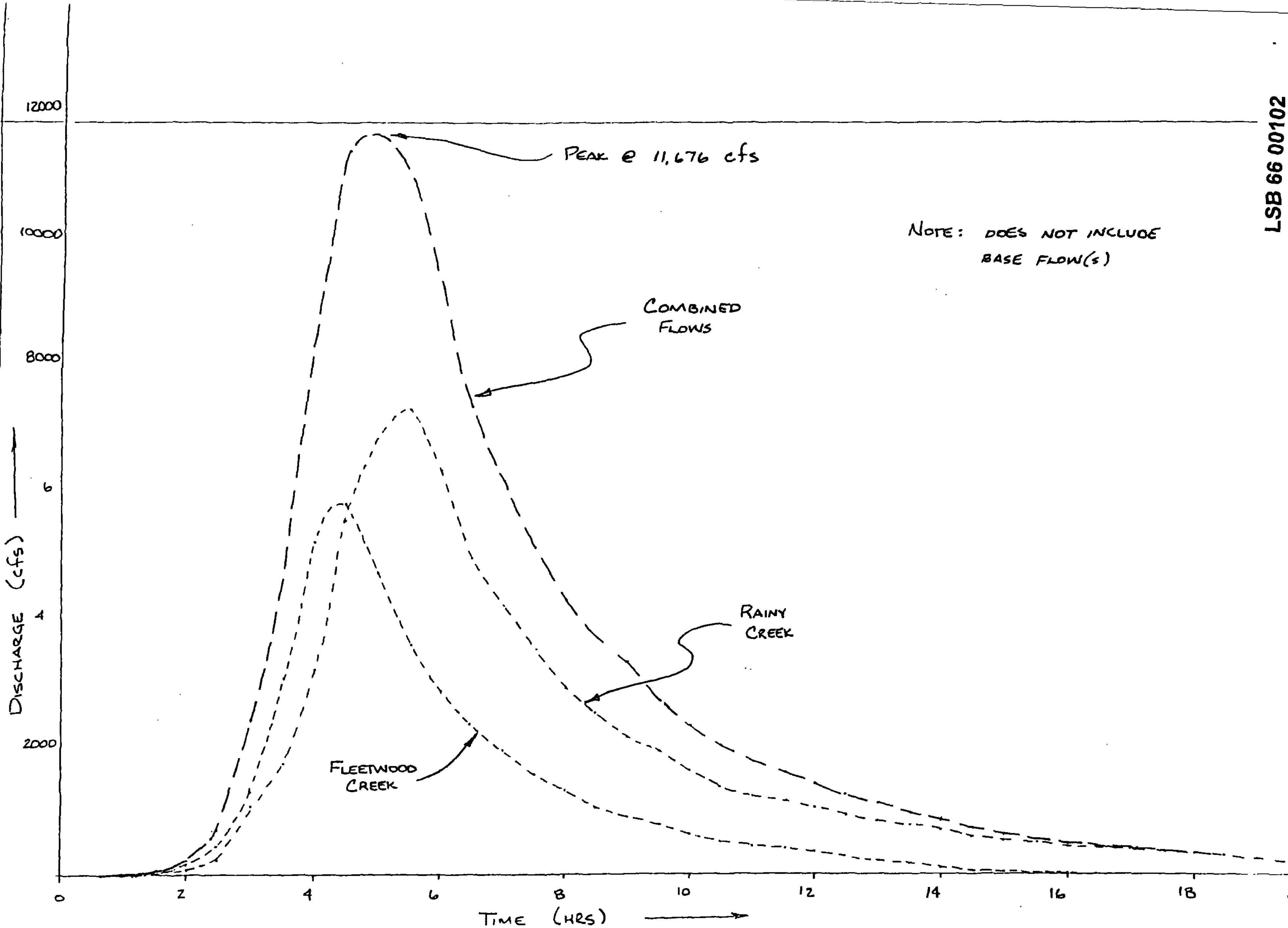
					10/
11.5	502	99.6	4500	18	439
12.0	524	99.9	4511	11	377
12.5	546	100.0	4517	7	322
13.0				0	270
13.5					213
14.0					169
14.5					117
15.0					72
					31
					18
					11
					6
					3


"S - graph" - FLEETWOOD CREEK
DISCHARGE (% ULTIMATE)
TIME (% of Lg)
0
20
40
60
80
100
0
50
100
150
200
250
300
350
400
450
5

LSB 66 00099

TIME (HRS)	RAINY CREEK (cfs)	FLEETWOOD CREEK (cfs)	COMBINED (cfs)
0	0	0	0
0.5	0	0	0
1.0	5	6	11
1.5	22	35	57
2.0	73	165	238
2.5	280	447	727
3.0	1131	1354	2485
3.5	1782	3112	4894
4.0	3242	5278	8520
4.5	5582	5884	11,466
5.0	6900	4776	11,676
5.5	7330	3652	10,982
6.0	6350	2917	9267
6.5	5012	2380	7392
7.0	4256	1956	6212
7.5	3560	1604	5164
8.0	2960	1350	4310
8.5	2606	1123	3729
9.0	2218	940	3158
9.5	1921	800	2721
10.0	1675	680	2355
10.5	1466	578	2044
11.0	1291	510	1801

TIME	RAINY CREEK	FLEETWOOD CK	COMBINED
11.5	1197	439	1636
12.0	1076	377	1453
12.5	963	322	1285
13.0	890	270	1160
13.5	804	213	1017
14.0	722	169	891
14.5	648	117	765
15.0	604	72	676
15.5	556	31	587
16.0	505	18	523
16.5	459	11	470
17.0	420	6	426
17.5	375	3	378
18.0	336	0	336
18.5	314		314
19.0	280		280
19.5	243		243
20.0	204		204
20.5	164		164
21.0	115		115
21.5	43		43
22.0	25		25
22.5	17		17
23.0	9		9


12000
10000
8000
6
4
2000
Discharge (cfs)
0
2
4
6
8
10
12
14
16
18
Time (hrs)
Peak @ 11,676 cfs
Note: does not include base flow(s)
Combined Flows
Rainy Creek
Fleetwood Creek

APPENDIX C

CONTROL STRUCTURE and EMERGENCY SPILLWAY CALCULATIONS

CONTROL STRUCTURE

CULVERT RATING CURVE

SINGLE 4' X 8' CONCRETE BOX CULVERT

ELEVATION	INLET CONTROL			
	HW (ft)	HW/D	Q/B (cfs)	Q (cfs)
2900	0	0	0	0
2901	1.0	0.25	3.5	28
2902	2.0	0.50	7.2	58
2905	5.0	1.25	28	224
2907	7.0	1.75	40	320
2910	10.0	2.5	52	416
2915	15.0	3.75	68	544
2920	20.0	5.0	82	656
2926	26.0	6.5	93	744

CONTROL STRUCTURE

CULVERT RATING CURVE

TWIN 4' X 6' CONCRETE BOX CULVERTS

ELEVATION	INLET CONTROL				
	HW (ft)	HW/D	Q/B (cfs)	Q/Box (cfs)	Q_{Total} (cfs)
2900	0	0	0	0	0
2901	1.0	0.25	3.5	21	42
2902	2.0	0.50	7	42	84
2905	5.0	1.25	27	162	324
2907	7.0	1.75	38	228	456
2910	10.0	2.5	48	288	576
2915	15.0	3.75	65	390	780
2920	20.0	5.0	78	468	936
2926	26.0	6.5	90	540	1080

EMERGENCY SPILLWAY

STAGE-DISCHARGE

50 FT EMERGENCY SPILLWAY, CREST ELEVATION 2922.0

STAGE-DISCHARGE FOR EMERGENCY SPILLWAY			
ELEVATION	H_p	$Q_{High\ Flow}$	Q_{Total}
2922	---	---	990
2923	1.0	200	1223
2924	2.0	350	1386
2925	3.0	650	1709
2926	4.0	1000	2080

APPENDIX D

FLOOD ROUTING RESULTS

FLOOD ROUTING: 100-YEAR, 24-HOUR EVENT

TWIN 4' X 6' BOX CULVERTS

ESERVOIR ROUTING PROG. (RES.BAS) SMH.3-06-87

WR GRACE DAM
100 YR 24 HR
LLB
12-15-1991

INPUT CONTROLS:
NO OF STORAGE CURVE POINTS= 7 . DELTA T= .2

CASE I EMERG. SPLWY. CURVE, CREST LENGTH = 500 FT
EARTH EMERG. SPILLWAY: CREST EL.= 2900 , WIDTH= 12
SIDE SLOPE= .001 , EXIT SLOPE= .04

SPILLWAY DISCHARGE CURVES:
....
CTR: 1=HSW.2=HSO.3=BO.4=FB.10=LSW.20=LSO.100=ES.1000=CS
500=ES CURVE IS EXCEEDED. Q'S ARE EXTRAPOLATED

ELEV	PRIN. Q	CHUTE Q	! EMERG. SPLWY. VALUES Q/FT	EFF. W.	TOT. Q !	CTR
2900.00	0.00	0.00	0.00	12.00	0.00	100
2900.50	0.00	0.00	1.75	12.00	21.00	100
2901.00	0.00	0.00	3.50	12.00	42.00	100
2902.00	0.00	0.00	7.00	12.00	84.01	100
2904.00	0.00	0.00	20.33	12.00	244.05	100
2906.00	0.00	0.00	32.50	12.00	390.10	100
2907.00	0.00	0.00	38.00	12.00	456.14	100
2910.00	0.00	0.00	48.00	12.00	576.20	100
2915.00	0.00	0.00	65.00	12.01	780.33	100
2920.00	0.00	0.00	78.00	12.01	936.45	100
2926.00	0.00	0.00	90.00	12.01	1080.57	100

**** WARNING: DELTA T MAY BE TOO LARGE FOR PROPER ROUTING ****

INITIAL ROUTING ELEV= 2900.00 STORAGE= 26.40
ACTUAL DELTA T ROUTING INTERVAL= .2 HRS.. PRINTOUT INTERVAL= 1 HRS.
INFLOW Q INTEGRATED FROM TIME-Q DATA IN FILE: HYD100.

TIME INT.,HRS		INFLOW, CFS	S/T+O/2	OUTFLOW, CFS	EXIT VEL
INITIAL			1597.20	0.00	

PEAK
ELEV= 2900.00 STORAGE= 26.37
TIME= 0.00 -0.20 INFLOW= -1.73 S/T O/2= 1595.47
TOTAL SPLWY DIS= 0.00 CTR= 0
PRIN Q= 0.00 CHUTE Q= 0.00
EMRG Q= 0.00 EMRG EXIT VEL= 0.00

0.80	-1.00	3.92	1599.91	0.24	0.69
1.80	-2.00	5.86	1621.99	2.17	1.68
2.80	-3.00	5.99	1637.91	3.57	2.05
3.80	-4.00	6.00	1648.11	4.46	2.24
4.80	-5.00	6.00	1654.56	5.03	2.35
5.80	-6.00	6.00	1658.65	5.38	2.42
6.80	-7.00	6.00	1661.23	5.61	2.46
7.80	-8.00	6.00	1662.86	5.75	2.48
8.80	-9.00	6.00	1663.89	5.84	2.50
9.80	-10.00	6.06	1664.60	5.91	2.51

12.80	-13.00	316.56	3483.72	140.53	8.91
13.80	-14.00	286.64	4159.90	194.24	10.14
14.80	-15.00	258.97	4477.29	219.45	10.65
15.80	-16.00	234.12	4577.95	227.45	10.81

PEAK

ELEV= 2903.80 STORAGE= 73.84

TIME= 16.00 -16.20 INFLOW= 230.43 S/T O/2= 4580.93

TOTAL SPLWY DIS= 227.69 CTR= 100

PRIN Q= 0.00 CHUTE Q= 0.00

EMRG Q= 227.69 EMRG EXIT VEL= 10.81

16.80	-17.00	219.14	4562.80	226.25	10.78
17.80	-18.00	207.91	4503.14	221.51	10.69
18.80	-19.00	195.51	4411.02	214.19	10.55
19.80	-20.00	183.75	4298.89	205.28	10.37
20.80	-21.00	176.84	4188.19	196.49	10.19
21.80	-22.00	167.59	4078.97	187.81	10.01
22.80	-23.00	162.72	3978.78	179.86	9.84
23.80	-24.00	156.54	3889.10	172.73	9.68
24.80	-25.00	151.36	3807.64	166.26	9.53
25.80	-26.00	104.43	3639.89	152.94	9.22
26.80	-27.00	40.20	3247.26	121.75	8.42
27.80	-28.00	15.30	2822.50	88.01	7.39
28.80	-29.00	8.32	2482.82	65.67	6.57
29.80	-30.00	6.46	2224.24	49.29	5.86
30.80	-31.00	6.06	2034.84	36.26	5.18
31.80	-32.00	6.06	1905.39	26.29	4.56
32.80	-33.00	6.06	1818.75	19.42	4.04
33.80	-34.00	6.06	1762.71	14.50	3.59
34.80	-35.00	6.06	1727.28	11.40	3.26
35.80	-36.00	6.06	1704.88	9.44	3.03
36.80	-37.00	6.06	1690.72	8.20	2.86
37.80	-38.00	6.06	1681.77	7.41	2.75
38.80	-39.00	6.06	1676.11	6.92	2.67
39.80	-40.00	6.06	1672.54	6.60	2.62
40.80	-41.00	6.06	1670.27	6.40	2.59
41.80	-42.00	6.06	1668.84	6.28	2.57

TOTAL VOLUME EMERG SPLWY FLOW= 273.43 AF

TOTAL VOLUME OF HYD ROUTED= 274.50 AF

FLOOD ROUTING: 0.5 PMF EVENT

TWIN 4' X 6' BOX CULVERTS

```
.ESERVOIR ROUTING PROG.  (RES.BAS)  SMH.3-06-87

  WR GRACE DAM
  .5PMP
  LLB
  12-14-1991

INPUT CONTROLS:
NO OF STORAGE CURVE POINTS=  7 .  DELTA T=  .2

 CASE I  EMERG. SPLWY. CURVE, CREST LENGTH = 500 FT
EARTH EMERG. SPILLWAY:  CREST EL.=  2900 . WIDTH=  12 '
      SIDE SLOPE=  .001 , EXIT SLOPE=  .04


 SPILLWAY DISCHARGE CURVES:
............................
CTR: 1=HSW.2=HSO.3=BO.4=FB.10=LSW.20=LSO.100=ES,1000=CS
     500=ES CURVE IS EXCEEDED, Q'S ARE EXTRAPOLATED
```

ELEV	PRIN. Q	CHUTE Q	Q/FT	EFF. W.	TOT. Q	CTR
			EMERG. SPLWY. VALUES			
2900.00	0.00	0.00	0.00	12.00	0.00	100
2900.50	0.00	0.00	1.75	12.00	21.00	100
2901.00	0.00	0.00	3.50	12.00	42.00	100
2902.00	0.00	0.00	7.00	12.00	84.01	100
2904.00	0.00	0.00	20.33	12.00	244.05	100
2906.00	0.00	0.00	32.50	12.00	390.10	100
2907.00	0.00	0.00	38.00	12.00	456.14	100
2910.00	0.00	0.00	48.00	12.00	576.20	100
2915.00	0.00	0.00	65.00	12.01	780.33	100
2920.00	0.00	0.00	78.00	12.01	936.45	100
2926.00	0.00	0.00	90.00	12.01	1080.57	100

```
 **** WARNING: DELTA T MAY BE TOO LARGE FOR PROPER ROUTING ****


INITIAL ROUTING ELEV=  2900.00   STORAGE=     26.40
ACTUAL DELTA T ROUTING INTERVAL=  .2  HRS.,  PRINTOUT INTERVAL=  .1  HRS.
INFLOW Q INTEGRATED FROM TIME-Q DATA IN FILE: HYD55.PMP
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TIME INT.,HRS		INFLOW, CFS	S/T+O/2	OUTFLOW, CFS	EXIT VEL
INITIAL			1597.20	0.00	
0.40	-0.60	3.41	1598.88	0.15	0.57
0.60	-0.80	5.89	1604.62	0.65	1.04
0.80	-1.00	7.02	1610.99	1.21	1.33
1.00	-1.20	7.55	1617.33	1.76	1.55
1.20	-1.40	7.79	1623.36	2.29	1.72
1.40	-1.60	7.90	1628.97	2.78	1.86
1.60	-1.80	7.96	1634.14	3.24	1.97
1.80	-2.00	7.98	1638.89	3.65	2.07
2.00	-2.20	7.99	1643.22	4.03	2.15
2.20	-2.40	8.00	1647.18	4.38	2.23
2.40	-2.60	8.00	1650.80	4.70	2.29
2.60	-2.80	8.00	1654.10	4.99	2.34
2.80	-3.00	8.00	1657.12	5.25	2.39
3.00	-3.20	8.00	1659.87	5.49	2.44
3.20	-3.40	8.00	1662.37	5.71	2.48
3.40	-3.60	8.00	1664.66	5.91	2.51
3.60	-3.80	8.00	1666.75	6.10	2.54

4.00	-4.20	8.00	1670.39	6.41	2.59
4.20	-4.40	8.00	1671.98	6.55	2.62
4.40	-4.60	8.00	1673.42	6.68	2.64
4.60	-4.80	8.00	1674.74	6.80	2.65
4.80	-5.00	8.00	1675.95	6.90	2.67
5.00	-5.20	8.00	1677.05	7.00	2.68
5.20	-5.40	8.00	1678.05	7.09	2.70
5.40	-5.60	8.00	1678.96	7.17	2.71
5.60	-5.80	8.00	1679.80	7.24	2.72
5.80	-6.00	8.00	1680.56	7.31	2.73
6.00	-6.20	8.01	1681.26	7.37	2.74
6.20	-6.40	8.04	1681.94	7.43	2.75
6.40	-6.60	8.19	1682.70	7.49	2.76
6.60	-6.80	8.77	1683.98	7.60	2.78
6.80	-7.00	10.35	1686.72	7.85	2.81
7.00	-7.20	13.74	1692.61	8.36	2.88
7.20	-7.40	19.77	1704.02	9.36	3.02
7.40	-7.60	29.09	1723.75	11.09	3.23
7.60	-7.80	42.21	1754.87	13.82	3.52
7.80	-8.00	59.39	1800.44	17.81	3.90
8.00	-8.20	80.74	1863.38	23.05	4.32
8.20	-8.40	106.27	1946.60	29.46	4.77
8.40	-8.60	135.87	2053.01	37.66	5.26
8.60	-8.80	169.87	2185.22	46.81	5.74
8.80	-9.00	209.31	2347.72	57.11	6.22
9.00	-9.20	256.38	2546.98	69.74	6.73
9.20	-9.40	313.96	2791.20	85.52	7.31
9.40	-9.60	385.43	3091.11	109.35	8.06
9.60	-9.80	475.67	3457.43	138.44	8.86
9.80	-10.00	597.13	3916.11	174.88	9.73
10.00	-10.20	790.91	4532.14	223.81	10.74
10.20	-10.40	1140.91	5449.24	272.70	11.62
10.40	-10.60	1738.37	6914.91	336.11	12.63
10.60	-10.80	2589.74	9168.53	429.72	13.94
10.80	-11.00	3554.47	12293.28	496.44	14.76
11.00	-11.20	4403.85	16200.69	560.62	15.50
11.20	-11.40	4954.91	20594.98	618.83	16.12
11.40	-11.60	5144.53	25120.68	674.83	16.69
11.60	-11.80	5015.65	29461.50	728.54	17.21
11.80	-12.00	4678.04	33411.00	777.41	17.66
12.00	-12.20	4253.34	36886.93	806.27	17.92
12.20	-12.40	3824.35	39905.00	830.44	18.14
12.40	-12.60	3429.79	42504.36	851.25	18.32
12.60	-12.80	3084.40	44737.51	869.13	18.47
12.80	-13.00	2790.65	46659.04	884.51	18.60
13.00	-13.20	2544.68	48319.20	897.80	18.71
13.20	-13.40	2340.83	49762.23	909.36	18.81
13.40	-13.60	2174.31	51027.18	919.49	18.89
13.60	-13.80	2039.95	52147.64	928.46	18.96
13.80	-14.00	1931.21	53150.40	936.47	19.03
14.00	-14.20	1840.70	54054.62	941.46	19.07
14.20	-14.40	1761.56	54874.71	945.98	19.11
14.40	-14.60	1688.89	55617.63	950.08	19.14
14.60	-14.80	1620.32	56287.87	953.78	19.17
14.80	-15.00	1555.92	56890.01	957.10	19.20
15.00	-15.20	1496.93	57429.84	960.07	19.22
15.20	-15.40	1444.30	57914.07	962.74	19.24
15.40	-15.60	1398.18	58349.51	965.14	19.26
15.60	-15.80	1358.21	58742.58	967.31	19.28
15.80	-16.00	1324.07	59099.34	969.28	19.29
16.00	-16.20	1295.51	59425.57	971.08	19.31
16.20	-16.40	1271.71	59726.20	972.74	19.32
16.40	-16.60	1251.14	60004.61	974.27	19.33
16.60	-16.80	1232.01	60262.35	975.69	19.34
16.80	-17.00	1212.81	60499.46	977.00	19.36

17.20	-17.40	1170.54	60907.28	979.25	19.37
17.40	-17.60	1147.33	61075.36	980.18	19.38
17.60	-17.80	1123.65	61218.84	980.97	19.39
17.80	-18.00	1100.68	61338.55	981.63	19.39
18.00	-18.20	1079.50	61436.43	982.17	19.40
18.20	-18.40	1060.39	61514.66	982.60	19.40
18.40	-18.60	1042.60	61574.66	982.93	19.40
18.60	-18.80	1024.93	61616.66	983.16	19.40
18.80	-19.00	1006.36	61639.86	983.29	19.40
19.00	-19.20	986.79	61643.37	983.31	19.40

PEAK
ELEV= 2921.95 STORAGE= 1010.77
TIME= 19.00 -19.20 INFLOW= 986.79 S/T O/2= 61643.37
TOTAL SPLWY DIS= 983.31 CTR= 100
PRIN Q= 0.00 CHUTE Q= 0.00
EMRG Q= 983.31 EMRG EXIT VEL= 19.40

19.20	-19.40	967.55	61627.61	983.22	19.40
19.40	-19.60	950.95	61595.34	983.04	19.40
19.60	-19.80	938.53	61550.83	982.80	19.40
19.80	-20.00	929.77	61497.80	982.51	19.40
20.00	-20.20	922.51	61437.80	982.17	19.40
20.20	-20.40	914.23	61369.86	981.80	19.39
20.40	-20.60	903.26	61291.32	981.37	19.39
20.60	-20.80	889.28	61199.23	980.86	19.39
20.80	-21.00	873.44	61091.82	980.27	19.38
21.00	-21.20	857.66	60969.21	979.59	19.38
21.20	-21.40	843.50	60833.12	978.84	19.37
21.40	-21.60	831.80	60686.08	978.03	19.36
21.60	-21.80	822.81	60530.86	977.17	19.36
21.80	-22.00	816.20	60369.89	976.29	19.35
22.00	-22.20	811.15	60204.75	975.37	19.34
22.20	-22.40	806.37	60035.74	974.44	19.33
22.40	-22.60	800.46	59861.76	973.48	19.33
22.60	-22.80	792.52	59680.80	972.49	19.32
22.80	-23.00	782.76	59491.08	971.44	19.31
23.00	-23.20	772.23	59291.86	970.34	19.30
23.20	-23.40	762.14	59083.66	969.19	19.29
23.40	-23.60	753.46	58867.93	968.00	19.28
23.60	-23.80	746.63	58646.56	966.78	19.27
23.80	-24.00	741.53	58421.30	965.54	19.26
24.00	-24.20	737.48	58193.24	964.28	19.25
24.20	-24.40	733.22	57962.18	963.01	19.24
24.40	-24.60	726.14	57725.30	961.70	19.23
24.60	-24.80	710.90	57474.50	960.32	19.22
24.80	-25.00	679.79	57193.98	958.77	19.21
25.00	-25.20	626.10	56861.30	956.94	19.20
25.20	-25.40	549.84	56454.20	954.69	19.18
25.40	-25.60	459.46	55958.97	951.96	19.16
25.60	-25.80	366.79	55373.80	948.74	19.13
25.80	-26.00	282.04	54707.11	945.06	19.10
26.00	-26.20	211.59	53973.64	941.01	19.07
26.20	-26.40	157.00	53189.62	936.69	19.03
26.40	-26.60	116.15	52369.08	930.23	18.98
26.60	-26.80	86.02	51524.87	923.47	18.92
26.80	-27.00	64.01	50665.41	916.59	18.87
27.00	-27.20	48.07	49796.89	909.64	18.81
27.20	-27.40	36.56	48923.81	902.65	18.75
27.40	-27.60	28.28	48049.44	895.65	18.69
27.60	-27.80	22.33	47176.13	888.65	18.64
27.80	-28.00	18.06	46305.53	881.68	18.58
28.00	-28.20	14.98	45438.83	874.74	18.52
28.20	-28.40	12.76	44576.85	867.84	18.46
28.40	-28.60	11.17	43720.17	860.98	18.40
28.60	-28.80	10.01	42869.20	854.17	18.34

29.00	-29.20	[illegible]	[illegible]	[illegible]	[illegible]
29.20	-29.40	8.38	40353.19	834.02	18.17
29.40	-29.60	8.24	39527.41	827.41	18.11
29.60	-29.80	8.20	38708.20	820.85	18.05
29.80	-30.00	8.20	37895.54	814.35	18.00
30.00	-30.20	8.20	37089.40	807.89	17.94
30.20	-30.40	8.20	36289.71	801.49	17.88
30.40	-30.60	8.20	35496.42	795.14	17.82
30.60	-30.80	8.20	34709.48	788.84	17.77
30.80	-31.00	8.20	33928.84	782.59	17.71
31.00	-31.20	8.20	33154.46	774.24	17.64
31.20	-31.40	8.20	32388.42	764.76	17.55
31.40	-31.60	8.20	31631.87	755.40	17.46
31.60	-31.80	8.20	30884.67	746.15	17.38
31.80	-32.00	8.20	30146.72	737.02	17.29
32.00	-32.20	8.20	29417.91	728.00	17.21
32.20	-32.40	8.20	28698.11	719.10	17.12
32.40	-32.60	8.20	27987.21	710.30	17.04
32.60	-32.80	8.20	27285.11	701.61	16.95
32.80	-33.00	8.20	26591.70	693.03	16.87
33.00	-33.20	8.20	25906.87	684.56	16.79
33.20	-33.40	8.20	25230.52	676.19	16.71
33.40	-33.60	8.20	24562.53	667.92	16.62
33.60	-33.80	8.20	23902.80	659.76	16.54
33.80	-34.00	8.20	23251.24	651.70	16.46
34.00	-34.20	8.20	22607.75	643.74	16.38
34.20	-34.40	8.20	21972.21	635.87	16.30
34.40	-34.60	8.20	21344.54	628.11	16.22
34.60	-34.80	8.20	20724.63	620.44	16.14
34.80	-35.00	8.20	20112.40	612.86	16.06
35.00	-35.20	8.20	19507.74	605.38	15.98
35.20	-35.40	8.20	18910.56	597.99	15.91
35.40	-35.60	8.20	18320.77	590.69	15.83
35.60	-35.80	8.20	17738.28	583.49	15.75
35.80	-36.00	8.20	17163.00	576.37	15.67
36.00	-36.20	8.20	16594.83	567.09	15.57
36.20	-36.40	8.20	16035.95	557.91	15.47
36.40	-36.60	8.20	15486.24	548.88	15.37
36.60	-36.80	8.20	14945.56	540.00	15.27
36.80	-37.00	8.20	14413.75	531.27	15.17
37.00	-37.20	8.20	13890.69	522.68	15.07
37.20	-37.40	8.20	13376.21	514.23	14.97
37.40	-37.60	8.20	12870.19	505.92	14.88
37.60	-37.80	8.20	12372.47	497.74	14.78
37.80	-38.00	8.20	11882.93	489.70	14.68
38.00	-38.20	8.20	11401.43	481.79	14.59
38.20	-38.40	8.20	10927.84	474.02	14.49
38.40	-38.60	8.20	10462.02	466.37	14.40
38.60	-38.80	8.20	10003.86	458.84	14.31
38.80	-39.00	8.20	9553.22	444.87	14.13
39.00	-39.20	8.20	9116.55	427.67	13.91
39.20	-39.40	8.20	8697.08	411.15	13.69
39.40	-39.60	8.20	8294.13	395.27	13.48
39.60	-39.80	8.20	7907.06	379.04	13.25
39.80	-40.00	8.20	7536.23	362.99	13.03
40.00	-40.20	8.20	7181.44	347.64	12.80
40.20	-40.40	8.20	6842.00	332.96	12.58
40.40	-40.60	8.20	6517.24	318.91	12.37
40.60	-40.80	8.20	6206.54	305.47	12.16
40.80	-41.00	8.20	5909.27	292.60	11.95
41.00	-41.20	8.20	5624.87	280.30	11.75
41.20	-41.40	6.64	5351.21	268.46	11.55
41.40	-41.60	5.09	5087.84	257.07	11.35

TOTAL VOLUME EMERG SPLWY FLOW= 2041.08 AF

FLOOD ROUTING: 0.5 PMF EVENT

SINGLE 4' X 8' BOX CULVERTS

RESERVOIR ROUTING PROG. (RES.BAS) SMH.3-06-87

WR GRACE DAM
.5PMP
LLB
01-01-1992

INPUT CONTROLS:
NO OF STORAGE CURVE POINTS= 7 . DELTA T= .2

CASE I EMERG. SPLWY. CURVE. CREST LENGTH = 500 FT
EARTH EMERG. SPILLWAY: CREST EL.= 2900 . WIDTH= 8 '
SIDE SLOPE= .001 . EXIT SLOPE= .04

SPILLWAY DISCHARGE CURVES:
....
CTR: 1=HSW.2=HSO.3=BO.4=FB.10=LSW.20=LSO.100=ES.1000=CS
500=ES CURVE IS EXCEEDED. Q'S ARE EXTRAPOLATED

			EMERG. SPLWY. VALUES			
ELEV	PRIN. Q	CHUTE Q	Q/FT	EFF. W.	TOT. Q	CTR
2900.00	0.00	0.00	0.00	8.00	0.00	100
2900.50	0.00	0.00	1.75	8.00	14.00	100
2901.00	0.00	0.00	3.50	8.00	28.00	100
2902.00	0.00	0.00	7.50	8.00	60.01	100
2904.00	0.00	0.00	21.17	8.00	169.38	100
2906.00	0.00	0.00	34.00	8.00	272.11	100
2907.00	0.00	0.00	40.00	8.00	320.15	100
2910.00	0.00	0.00	52.00	8.00	416.23	100
2915.00	0.00	0.00	68.00	8.01	544.36	100
2920.00	0.00	0.00	82.00	8.01	656.49	100
2926.00	0.00	0.00	93.00	8.01	744.60	100

**** WARNING: DELTA T MAY BE TOO LARGE FOR PROPER ROUTING ****

STORAGE INDICATION CURVE:
....

ELEV.	STORAGE	S/^T+O/2	TOT. DIS.	CTR.
2900.00	26.40	1597.20	0.00	100
2900.50	30.19*	1833.33	14.00	100
2901.00	34.52*	2102.29	28.00	100
2902.00	45.13*	2760.12	60.01	100
2904.00	77.11*	4749.58	169.38	100
2906.00	131.70	8103.91	272.11	100
2907.00	158.86*	9771.13	320.15	100
2910.00	278.70	17069.46	416.23	100
2915.00	549.70	33529.03	544.36	100
2920.00	870.70	53005.59	656.49	100
2926.00	1301.50	79113.05	744.60	100

*--VALUE INSERTED BY LOG-LOG INTERP BY PROG.

LSB 66 00114

INITIAL ROUTING ELEV= 2900.00 STORAGE= 26.40
ACTUAL DELTA T ROUTING INTERVAL= .2 HRS., PRINTOUT INTERVAL= 1 HRS.
INFLOW Q INTEGRATED FROM TIME-Q DATA IN FILE: HYD55.PMP

TIME	INT..HRS	INFLOW. CFS	S/T+O/2	OUTFLOW. CFS	EXIT VEL
	INITIAL		1597.20	0.00	
0.80	-1.00	7.02	1611.25	0.83	1.35

2.80	-3.00	[illegible]	[illegible]	[illegible]	[illegible]
3.80	-4.00	8.00	1683.42	5.11	2.78
4.80	-5.00	8.00	1696.25	5.87	2.94
5.80	-6.00	8.00	1705.70	6.43	3.05
6.80	-7.00	10.35	1715.93	7.04	3.17
7.80	-8.00	59.39	1836.73	14.18	4.19
8.80	-9.00	209.31	2421.51	43.53	6.56
9.80	-10.00	597.13	4094.01	133.34	10.26
10.80	-11.00	3554.47	12844.70	360.61	15.28
11.80	-12.00	4678.04	34808.40	551.72	18.11
12.80	-13.00	2790.65	49244.00	634.83	19.16
13.80	-14.00	1931.21	57011.92	670.01	19.57
14.80	-15.00	1555.92	62092.45	687.16	19.77
15.80	-16.00	1324.07	65652.80	699.17	19.91
16.80	-17.00	1212.81	68400.82	708.45	20.01
17.80	-18.00	1100.68	70577.73	715.79	20.10
18.80	-19.00	1006.36	72200.94	721.27	20.16
19.80	-20.00	929.77	73359.84	725.18	20.20
20.80	-21.00	873.44	74230.32	728.12	20.23
21.80	-22.00	816.20	74757.77	729.90	20.25
22.80	-23.00	782.76	75098.97	731.05	20.27
23.80	-24.00	741.53	75218.61	731.46	20.27

PEAK

ELEV= 2925.11 STORAGE= 1237.37

TIME= 24.20 -24.40 INFLOW= 733.22 S/T O/2= 75226.38

TOTAL SPLWY DIS= 731.48 CTR= 100

PRIN Q= 0.00 CHUTE Q= 0.00

EMRG Q= 731.48 EMRG EXIT VEL= 20.27

24.80	-25.00	679.79	75148.87	731.22	20.27
25.80	-26.00	282.04	73783.30	726.61	20.22
26.80	-27.00	64.01	70803.95	716.56	20.11
27.80	-28.00	18.06	67397.27	705.06	19.98
28.80	-29.00	9.20	63953.38	693.44	19.84
29.80	-30.00	8.20	60550.93	681.95	19.71
30.80	-31.00	8.20	57204.84	670.66	19.58
31.80	-32.00	8.20	53914.84	659.56	19.45
32.80	-33.00	8.20	50687.37	643.14	19.26
33.80	-34.00	8.20	47549.02	625.07	19.04
34.80	-35.00	8.20	44499.98	607.52	18.82
35.80	-36.00	8.20	41537.71	590.46	18.61
36.80	-37.00	8.20	38659.73	573.89	18.40
37.80	-38.00	8.20	35863.64	557.80	18.19
38.80	-39.00	8.20	33147.13	541.38	17.97
39.80	-40.00	8.20	30522.41	520.95	17.70
40.80	-41.00	8.20	27998.26	501.30	17.43

TOTAL VOLUME EMERG SPLWY FLOW= 1690.60 AF

TOTAL VOLUME OF HYD ROUTED= 2094.52 AF

FLOOD ROUTING: 0.55 PMF, NO EMERGENCY SPILLWAY

TWIN 4' X 6' BOX CULVERTS

ESERVOIR ROUTING PROG. (RES.BAS) SMH.3-06-87

WR GRACE DAM
FBD
LLB
12-14-1991

INPUT CONTROLS:
NO OF STORAGE CURVE POINTS= 7 , DELTA T= .2

CASE I EMERG. SPLWY. CURVE, CREST LENGTH = 500 FT
EARTH EMERG. SPILLWAY: CREST EL.= 2900 , WIDTH= 12
SIDE SLOPE= .001 , EXIT SLOPE= .04

SPILLWAY DISCHARGE CURVES:
....
CTR: 1=HSW,2=HSO.3=BO.4=FB,10=LSW,20=LSO.100=ES.1000=CS
500=ES CURVE IS EXCEEDED, Q'S ARE EXTRAPOLATED

			EMERG. SPLWY. VALUES			
ELEV	PRIN. Q	CHUTE Q	Q/FT	EFF. W.	TOT. Q	CTR
2900.00	0.00	0.00	0.00	12.00	0.00	100
2900.50	0.00	0.00	1.75	12.00	21.00	100
2901.00	0.00	0.00	3.50	12.00	42.00	100
2902.00	0.00	0.00	7.00	12.00	84.01	100
2904.00	0.00	0.00	20.33	12.00	244.05	100
2906.00	0.00	0.00	32.50	12.00	390.10	100
2907.00	0.00	0.00	38.00	12.00	456.14	100
2910.00	0.00	0.00	48.00	12.00	576.20	100
2915.00	0.00	0.00	65.00	12.01	780.33	100
2920.00	0.00	0.00	78.00	12.01	936.45	100
2926.00	0.00	0.00	90.00	12.01	1080.57	100

**** WARNING: DELTA T MAY BE TOO LARGE FOR PROPER ROUTING ****

STORAGE INDICATION CURVE:
....

ELEV.	STORAGE	S/^T+O/2	TOT. DIS.	CTR.
2900.00	26.40	1597.20	0.00	100
2900.50	30.19*	1836.83	21.00	100
2901.00	34.52*	2109.29	42.00	100
2902.00	45.13*	2772.11	84.01	100
2904.00	77.11*	4786.91	244.05	100
2906.00	131.70	8162.90	390.10	100
2907.00	158.86*	9839.12	456.14	100
2910.00	278.70	17149.45	576.20	100
2915.00	549.70	33647.02	780.33	100
2920.00	870.70	53145.57	936.45	100
2926.00	1301.50	79281.03	1080.57	100

*--VALUE INSERTED BY LOG-LOG INTERP BY PROG.

LSB 66 00116

INITIAL ROUTING ELEV= 2900.00 STORAGE= 26.40
ACTUAL DELTA T ROUTING INTERVAL= .2 HRS., PRINTOUT INTERVAL= 1 HRS.
INFLOW Q INTEGRATED FROM TIME-Q DATA IN FILE: HYD5.PMP

TIME INT..HRS		INFLOW, CFS	S/T+O/2	OUTFLOW. CFS	EXIT VEL
INITIAL			1597.20	0.00	
0.80	-1.00	7.90	1612.89	1.37	1.40

2.80	-3.00	9.00	[illegible]	[illegible]	[illegible]
3.80	-4.00	9.00	1677.63	7.05	2.69
4.80	-5.00	9.00	1685.82	7.77	2.80
5.80	-6.00	9.10	1691.12	8.23	2.86
6.80	-7.00	27.25	1726.99	11.37	3.26
7.80	-8.00	127.39	2030.68	35.94	5.17
8.80	-9.00	331.99	2953.57	98.42	7.73
9.80	-10.00	823.78	5092.35	257.26	11.35
10.80	-11.00	4434.61	15773.85	553.61	15.42
11.80	-12.00	5695.06	42049.00	847.60	18.29
12.80	-13.00	3341.43	58474.15	965.83	19.27
13.80	-14.00	2287.57	66664.52	1011.00	19.62
14.80	-15.00	1832.04	71542.82	1037.90	19.83
15.80	-16.00	1553.62	74566.58	1054.57	19.96
16.80	-17.00	1419.73	76608.12	1065.83	20.04
17.80	-18.00	1286.09	77967.94	1073.33	20.10
18.80	-19.00	1174.05	78678.52	1077.25	20.13
19.80	-20.00	1083.26	78853.98	1078.21	20.13

PEAK
ELEV= 2925.90 STORAGE= 1294.46
TIME= 19.80 -20.00 INFLOW= 1083.26 S/T O/2= 78853.98
TOTAL SPLWY DIS= 1078.21 CTR= 100
PRIN Q= 0.00 CHUTE Q= 0.00
EMRG Q= 1078.21 EMRG EXIT VEL= 20.13

20.80	-21.00	1016.47	78706.13	1077.40	20.13
21.80	-22.00	948.88	78176.46	1074.48	20.11
22.80	-23.00	909.18	77451.48	1070.48	20.08
23.80	-24.00	860.57	76492.83	1065.19	20.04
24.80	-25.00	788.35	75340.38	1058.84	19.99
25.80	-26.00	326.83	72717.78	1044.38	19.88
26.80	-27.00	73.96	68277.61	1019.89	19.69
27.80	-28.00	20.66	63408.07	993.04	19.48
28.80	-29.00	10.39	58562.54	966.32	19.27
29.80	-30.00	9.23	53830.38	940.22	19.06
30.80	-31.00	9.23	49242.63	905.20	18.77
31.80	-32.00	9.23	44833.97	869.90	18.48
32.80	-33.00	9.23	40599.00	835.99	18.19
33.80	-34.00	9.23	36530.88	803.42	17.90
34.80	-35.00	9.23	32624.14	767.67	17.58
35.80	-36.00	9.23	28924.63	721.90	17.15
36.80	-37.00	9.23	25448.40	678.89	16.73
37.80	-38.00	9.23	22181.99	638.47	16.33
38.80	-39.00	9.23	19112.71	600.49	15.93
39.80	-40.00	9.23	16230.13	561.10	15.51
40.80	-41.00	9.23	13559.96	517.25	15.01

TOTAL VOLUME EMERG SPLWY FLOW= 2356.07 AF
TOTAL VOLUME OF HYD ROUTED= 2520.77 AF

FLOOD ROUTING: 0.66 PMF W/ EMERGENCY SPILLWAY

TWIN 4' X 6' BOX CULVERTS

ESERVOIR ROUTING PROG. (RES.BAS) SMH.3-06-87

WR GRACE DAM
.66PMP
LLB
12-14-1991

INPUT CONTROLS:
NO OF STORAGE CURVE POINTS= 7 . DELTA T= .2

CASE I EMERG. SPLWY. CURVE, CREST LENGTH = 100 FT
EARTH EMERG. SPILLWAY: CREST EL.= 2900 . WIDTH= 12 '
SIDE SLOPE= .01 . EXIT SLOPE= .04

SPILLWAY DISCHARGE CURVES:
....
CTR: 1=HSW.2=HSO.3=BO.4=FB.10=LSW.20=LSO.100=ES.1000=CS
500=ES CURVE IS EXCEEDED, Q'S ARE EXTRAPOLATED

			EMERG. SPLWY. VALUES			
ELEV	PRIN. Q	CHUTE Q	Q/FT	EFF. W.	TOT. Q	CTR
2900.00	0.00	0.00	0.00	12.00	0.00	100
2900.50	0.00	0.00	1.75	12.00	21.01	100
2901.00	0.00	0.00	3.50	12.01	42.03	100
2902.00	0.00	0.00	7.00	12.01	84.08	100
2904.00	0.00	0.00	20.33	12.02	244.48	100
2906.00	0.00	0.00	32.50	12.03	391.04	100
2907.00	0.00	0.00	38.00	12.04	457.35	100
2910.00	0.00	0.00	48.00	12.04	577.99	100
2915.00	0.00	0.00	65.00	12.05	783.30	100
2920.00	0.00	0.00	78.00	12.06	940.48	100
2926.00	0.00	0.00	173.30	12.10	2096.54	100

**** WARNING: DELTA T MAY BE TOO LARGE FOR PROPER ROUTING ****

STORAGE INDICATION CURVE:
....

ELEV.	STORAGE	S/^T+O/2	TOT. DIS.	CTR.
2900.00	26.40	1597.20	0.00	100
2900.50	30.19*	1836.84	21.01	100
2901.00	34.52*	2109.30	42.03	100
2902.00	45.13*	2772.15	84.08	100
2904.00	77.11*	4787.12	244.48	100
2906.00	131.70	8163.37	391.04	100
2907.00	158.86*	9839.73	457.35	100
2910.00	278.70	17150.35	577.99	100
2915.00	549.70	33648.50	783.30	100
2920.00	870.70	53147.59	940.48	100
2926.00	1301.50	79789.02	2096.54	100

*--VALUE INSERTED BY LOG-LOG INTERP BY PROG.

LSB 66 00118

INITIAL ROUTING ELEV= 2900.00 STORAGE= 26.40
ACTUAL DELTA T ROUTING INTERVAL= .2 HRS.. PRINTOUT INTERVAL= .1 HRS.
INFLOW Q INTEGRATED FROM TIME-Q DATA IN FILE: HYD75.PMP

TIME INT..HRS		INFLOW, CFS	S/T+O/2	OUTFLOW. CFS	EXIT VEL
INITIAL			1597.20	0.00	
0.40	-0.60	3.89	1599.28	0.18	0.62

1.00	-1.20	8.49	1620.00	2.00	1.63
1.20	-1.40	8.76	1626.77	2.59	1.80
1.40	-1.60	8.89	1633.07	3.14	1.95
1.60	-1.80	8.95	1638.87	3.65	2.07
1.80	-2.00	8.98	1644.19	4.12	2.17
2.00	-2.20	8.99	1649.06	4.55	2.26
2.20	-2.40	8.99	1653.51	4.94	2.33
2.40	-2.60	9.00	1657.57	5.29	2.40
2.60	-2.80	9.00	1661.28	5.62	2.46
2.80	-3.00	9.00	1664.66	5.91	2.51
3.00	-3.20	9.00	1667.75	6.18	2.56
3.20	-3.40	9.00	1670.56	6.43	2.60
3.40	-3.60	9.00	1673.13	6.66	2.63
3.60	-3.80	9.00	1675.47	6.86	2.66
3.80	-4.00	9.00	1677.61	7.05	2.69
4.00	-4.20	9.00	1679.56	7.22	2.72
4.20	-4.40	9.00	1681.34	7.38	2.74
4.40	-4.60	9.00	1682.97	7.52	2.76
4.60	-4.80	9.00	1684.45	7.65	2.78
4.80	-5.00	9.01	1685.80	7.77	2.80
5.00	-5.20	9.04	1687.08	7.88	2.82
5.20	-5.40	9.18	1688.38	7.99	2.83
5.40	-5.60	9.63	1690.01	8.14	2.85
5.60	-5.80	10.89	1692.77	8.38	2.89
5.80	-6.00	13.87	1698.26	8.86	2.95
6.00	-6.20	19.71	1709.11	9.81	3.07
6.20	-6.40	29.40	1728.70	11.53	3.28
6.40	-6.60	43.39	1760.56	14.32	3.57
6.60	-6.80	61.69	1807.93	18.47	3.96
6.80	-7.00	83.93	1873.39	23.83	4.38
7.00	-7.20	109.61	1959.17	30.44	4.83
7.20	-7.40	138.44	2067.17	38.78	5.32
7.40	-7.60	170.41	2198.80	47.70	5.78
7.60	-7.80	205.78	2356.88	57.73	6.24
7.80	-8.00	244.94	2544.09	69.61	6.73
8.00	-8.20	288.29	2762.77	83.49	7.23
8.20	-8.40	336.09	3015.37	103.44	7.88
8.40	-8.60	388.47	3300.40	126.13	8.53
8.60	-8.80	446.26	3620.53	151.61	9.18
8.80	-9.00	511.57	3980.49	180.27	9.84
9.00	-9.20	588.27	4388.48	212.74	10.51
9.20	-9.40	681.31	4857.05	247.51	11.17
9.40	-9.60	796.21	5405.75	271.33	11.59
9.60	-9.80	940.31	6074.74	300.37	12.07
9.80	-10.00	1131.76	6906.12	336.46	12.63
10.00	-10.20	1430.46	8000.12	383.95	13.31
10.20	-10.40	1959.12	9575.28	446.89	14.14
10.40	-10.60	2850.23	11978.63	492.65	14.70
10.60	-10.80	4108.81	15594.78	552.32	15.39
10.80	-11.00	5520.77	20563.23	620.47	16.12
11.00	-11.20	6742.67	26685.44	696.65	16.88
11.20	-11.40	7504.32	33493.10	781.37	17.67
11.40	-11.60	7719.71	40431.44	837.98	18.17
11.60	-11.80	7462.33	47055.80	891.37	18.63
11.80	-12.00	6902.97	53067.39	939.83	19.02
12.00	-12.20	6226.64	58354.21	1166.41	20.74
12.20	-12.40	5556.76	62744.56	1356.92	22.02
12.40	-12.60	4948.50	66336.14	1512.77	23.00
12.60	-12.80	4420.90	69244.27	1638.97	23.74
12.80	-13.00	3975.42	71580.72	1740.35	24.32
13.00	-13.20	3604.71	73445.08	1821.25	24.76
13.20	-13.40	3299.09	74922.91	1885.38	25.11
13.40	-13.60	3050.34	76087.87	1935.93	25.37
13.60	-13.80	2850.06	77001.99	1975.60	25.58

14.20	-14.40	2437.31	78669.18	2047.95	25.95
14.40	-14.60	2330.93	78952.16	2060.23	26.01
14.60	-14.80	2231.27	79123.20	2067.65	26.04
14.80	-15.00	2138.20	79193.75	2070.71	26.06

PEAK

ELEV= 2925.87 STORAGE= 1291.87
TIME= 14.80 -15.00 INFLOW= 2138.20 S/T O/2= 79193.75
TOTAL SPLWY DIS= 2070.71 CTR= 100
PRIN Q= 0.00 CHUTE Q= 0.00
EMRG Q= 2070.71 EMRG EXIT VEL= 26.06

15.00	-15.20	2053.33	79176.37	2069.95	26.06
15.20	-15.40	1977.85	79084.26	2065.96	26.04
15.40	-15.60	1911.89	78930.19	2059.27	26.00
15.60	-15.80	1854.78	78725.70	2050.40	25.96
15.80	-16.00	1805.97	78481.27	2039.79	25.90
16.00	-16.20	1765.03	78206.51	2027.87	25.84
16.20	-16.40	1730.77	77909.41	2014.98	25.78
16.40	-16.60	1701.07	77595.50	2001.36	25.71
16.60	-16.80	1673.48	77267.63	1987.13	25.64
16.80	-17.00	1645.91	76926.41	1972.32	25.56
17.00	-17.20	1616.91	76571.01	1956.90	25.48
17.20	-17.40	1585.88	76199.99	1940.80	25.40
17.40	-17.60	1553.21	75812.40	1923.98	25.31
17.60	-17.80	1519.99	75408.41	1906.45	25.22
17.80	-18.00	1487.82	74989.78	1888.28	25.12
18.00	-18.20	1458.14	74559.64	1869.62	25.02
18.20	-18.40	1431.32	74121.34	1850.60	24.92
18.40	-18.60	1406.37	73677.12	1831.32	24.82
18.60	-18.80	1381.63	73227.43	1811.81	24.71
18.80	-19.00	1355.76	72771.38	1792.02	24.60
19.00	-19.20	1328.60	72307.95	1771.91	24.49
19.20	-19.40	1301.93	71837.97	1751.52	24.38
19.40	-19.60	1278.85	71365.30	1731.01	24.27
19.60	-19.80	1261.43	70895.72	1710.63	24.15
19.80	-20.00	1248.97	70434.06	1690.60	24.04
20.00	-20.20	1238.56	69982.03	1670.98	23.93
20.20	-20.40	1226.83	69537.88	1651.71	23.82
20.40	-20.60	1211.51	69097.68	1632.61	23.71
20.60	-20.80	1192.18	68657.26	1613.49	23.60
20.80	-21.00	1170.41	68214.17	1594.27	23.48
21.00	-21.20	1148.72	67768.63	1574.93	23.37
21.20	-21.40	1129.24	67322.93	1555.59	23.26
21.40	-21.60	1113.08	66880.42	1536.39	23.14
21.60	-21.80	1100.57	66444.60	1517.48	23.03
21.80	-22.00	1091.27	66018.39	1498.99	22.92
22.00	-22.20	1084.06	65603.47	1480.98	22.81
22.20	-22.40	1077.25	65199.74	1463.46	22.70
22.40	-22.60	1068.94	64805.22	1446.34	22.59
22.60	-22.80	1057.94	64416.81	1429.49	22.49
22.80	-23.00	1044.51	64031.84	1412.78	22.38
23.00	-23.20	1030.07	63649.13	1396.17	22.28
23.20	-23.40	1016.24	63269.19	1379.69	22.17
23.40	-23.60	1004.30	62893.80	1363.40	22.07
23.60	-23.80	994.85	62525.25	1347.41	21.96
23.80	-24.00	987.71	62165.55	1331.80	21.86
24.00	-24.20	981.98	61815.74	1316.62	21.76
24.20	-24.40	975.99	61475.11	1301.84	21.66
24.40	-24.60	966.25	61139.53	1287.28	21.57
24.60	-24.80	945.68	60797.93	1272.45	21.47
24.80	-25.00	904.01	60429.49	1256.46	21.36
25.00	-25.20	832.32	60005.34	1238.06	21.23
25.20	-25.40	730.63	59497.91	1216.04	21.08
25.40	-25.60	610.20	58892.08	1189.75	20.90

26.00	-26.20	280.09	56558.67	1088.50	20.17
26.20	-26.40	207.39	55677.57	1050.26	19.89
26.40	-26.60	153.00	54780.31	1011.33	19.59
26.60	-26.80	112.88	53881.86	972.34	19.28
26.80	-27.00	83.58	52993.10	939.23	19.02
27.00	-27.20	62.34	52116.21	932.16	18.96
27.20	-27.40	47.02	51231.07	925.03	18.90
27.40	-27.60	36.00	50342.04	917.86	18.85
27.60	-27.80	28.08	49452.25	910.69	18.79
27.80	-28.00	22.39	48563.95	903.53	18.73
28.00	-28.20	18.29	47678.71	896.40	18.67
28.20	-28.40	15.34	46797.66	889.29	18.61
28.40	-28.60	13.22	45921.58	882.23	18.55
28.60	-28.80	11.68	45051.03	875.22	18.49
28.80	-29.00	10.60	44186.41	868.25	18.43
29.00	-29.20	9.89	43328.06	861.33	18.37
29.20	-29.40	9.50	42476.23	854.46	18.32
29.40	-29.60	9.32	41631.09	847.65	18.26
29.60	-29.80	9.27	40792.71	840.89	18.20
29.80	-30.00	9.27	39961.09	834.19	18.14
30.00	-30.20	9.27	39136.17	827.54	18.08
30.20	-30.40	9.27	38317.90	820.94	18.03
30.40	-30.60	9.27	37506.23	814.40	17.97
30.60	-30.80	9.27	36701.09	807.91	17.91
30.80	-31.00	9.27	35902.45	801.47	17.85
31.00	-31.20	9.27	35110.25	795.09	17.80
31.20	-31.40	9.27	34324.43	788.75	17.74
31.40	-31.60	9.27	33544.95	782.02	17.68
31.60	-31.80	9.27	32772.20	772.40	17.59
31.80	-32.00	9.27	32009.07	762.90	17.51
32.00	-32.20	9.27	31255.43	753.52	17.42
32.20	-32.40	9.27	30511.18	744.26	17.33
32.40	-32.60	9.27	29776.18	735.12	17.25
32.60	-32.80	9.27	29050.34	726.08	17.16
32.80	-33.00	9.27	28333.52	717.16	17.08
33.00	-33.20	9.27	27625.63	708.35	17.00
33.20	-33.40	9.27	26926.54	699.65	16.91
33.40	-33.60	9.27	26236.16	691.06	16.83
33.60	-33.80	9.27	25554.37	682.58	16.75
33.80	-34.00	9.27	24881.06	674.20	16.66
34.00	-34.20	9.27	24216.13	665.92	16.58
34.20	-34.40	9.27	23559.47	657.75	16.50
34.40	-34.60	9.27	22910.99	649.68	16.42
34.60	-34.80	9.27	22270.58	641.71	16.34
34.80	-35.00	9.27	21638.13	633.84	16.26
35.00	-35.20	9.27	21013.56	626.07	16.18
35.20	-35.40	9.27	20396.76	618.39	16.10
35.40	-35.60	9.27	19787.63	610.81	16.02
35.60	-35.80	9.27	19186.09	603.33	15.94
35.80	-36.00	9.27	18592.03	595.93	15.86
36.00	-36.20	9.27	18005.36	588.63	15.79
36.20	-36.40	9.27	17426.00	581.42	15.71
36.40	-36.60	9.27	16853.84	573.10	15.62
36.60	-36.80	9.27	16290.01	563.80	15.52
36.80	-37.00	9.27	15735.48	554.64	15.42
37.00	-37.20	9.27	15190.11	545.64	15.31
37.20	-37.40	9.27	14653.73	536.79	15.22
37.40	-37.60	9.27	14126.21	528.09	15.12
37.60	-37.80	9.27	13607.39	519.53	15.02
37.80	-38.00	9.27	13097.13	511.11	14.92
38.00	-38.20	9.27	12595.29	502.82	14.82
38.20	-38.40	9.27	12101.74	494.68	14.73
38.40	-38.60	9.27	11616.33	486.67	14.63
38.60	-38.80	9.27	11138.92	478.79	14.54

LSB 66 0012'

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
39.20	-39.40	9.27	9753.48	453.94	14.23
39.40	-39.60	9.27	9308.81	436.35	14.01
39.60	-39.80	9.27	8881.72	419.46	13.79
39.80	-40.00	9.27	8471.54	403.23	13.57
40.00	-40.20	9.27	8077.58	387.32	13.36
40.20	-40.40	9.27	7699.53	370.91	13.13
40.40	-40.60	9.27	7337.89	355.21	12.90
40.60	-40.80	9.27	6991.95	340.19	12.68
40.80	-41.00	9.27	6661.03	325.82	12.47
41.00	-41.20	9.26	6344.47	312.08	12.25

TOTAL VOLUME EMERG SPLWY FLOW= 2947.67 AF
TOTAL VOLUME OF HYD ROUTED= 3020.96 AF

INLET CHANNEL HYDRAULICS

COOPERATOR OR PROJECT= WR GRACE

DESCRIPTION= INLET CHANNEL

CALCULATED BY LLB

12-14-1991

LSB 66 00123

DITCH HYDRAULICS PROGRAM

DEPTH FT.	AREA SQ.FT.	B.W. FT.	AVE. S.S.	CHAN. GRADE	MAN. N	VEL. F.P.S.	FLOW CFS
1.0	12.00	10.0	2.0 TO 1	.00300	.0350	2.05	24.60
1.1	13.42	10.0	2.0 TO 1	.00300	.0350	2.17	29.12
1.2	14.88	10.0	2.0 TO 1	.00300	.0350	2.28	33.93
1.3	16.38	10.0	2.0 TO 1	.00300	.0350	2.39	39.15
1.4	17.92	10.0	2.0 TO 1	.00300	.0350	2.48	44.44
1.5	19.50	10.0	2.0 TO 1	.00300	.0350	2.58	50.31
1.6	21.12	10.0	2.0 TO 1	.00300	.0350	2.67	56.39
1.7	22.78	10.0	2.0 TO 1	.00300	.0350	2.76	62.87
1.8	24.48	10.0	2.0 TO 1	.00300	.0350	2.85	69.77
1.9	26.22	10.0	2.0 TO 1	.00300	.0350	2.94	77.09
2.0	28.00	10.0	2.0 TO 1	.00300	.0350	3.02	84.56
2.1	29.82	10.0	2.0 TO 1	.00300	.0350	3.10	92.44
2.2	31.68	10.0	2.0 TO 1	.00300	.0350	3.18	100.74
2.3	33.58	10.0	2.0 TO 1	.00300	.0350	3.26	109.47
2.4	35.52	10.0	2.0 TO 1	.00300	.0350	3.33	118.28
2.5	37.50	10.0	2.0 TO 1	.00300	.0350	3.40	127.50
2.6	39.52	10.0	2.0 TO 1	.00300	.0350	3.48	137.53
2.7	41.58	10.0	2.0 TO 1	.00300	.0350	3.54	147.19
2.8	43.68	10.0	2.0 TO 1	.00300	.0350	3.62	158.12
2.9	45.82	10.0	2.0 TO 1	.00300	.0350	3.68	168.62
3.0	48.00	10.0	2.0 TO 1	.00300	.0350	3.75	180.00
3.1	50.22	10.0	2.0 TO 1	.00300	.0350	3.81	191.34
3.2	52.48	10.0	2.0 TO 1	.00300	.0350	3.89	204.15

DITCH HYDRAULICS PROGRAM

DEPTH FT.	AREA SQ.FT.	B.W. FT.	AVE. S.S.	CHAN. GRADE	MAN. N	VEL. F.P.S.	FLOW CFS
3.4	57.12	10.0	2.0 TO 1	.00300	.0350	4.02	229.62
3.5	59.50	10.0	2.0 TO 1	.00300	.0350	4.08	242.76
3.6	61.92	10.0	2.0 TO 1	.00300	.0350	4.13	255.73
3.7	64.38	10.0	2.0 TO 1	.00300	.0350	4.19	269.75
3.8	66.88	10.0	2.0 TO 1	.00300	.0350	4.26	284.91
3.9	69.42	10.0	2.0 TO 1	.00300	.0350	4.32	299.89
4.0	72.00	10.0	2.0 TO 1	.00300	.0350	4.38	315.36
4.1	74.62	10.0	2.0 TO 1	.00300	.0350	4.43	330.57
4.2	77.28	10.0	2.0 TO 1	.00300	.0350	4.50	347.76
4.3	79.98	10.0	2.0 TO 1	.00300	.0350	4.56	364.71
4.4	82.72	10.0	2.0 TO 1	.00300	.0350	4.61	381.34
4.5	85.50	10.0	2.0 TO 1	.00300	.0350	4.67	399.29
4.6	88.32	10.0	2.0 TO 1	.00300	.0350	4.72	416.87
4.7	91.18	10.0	2.0 TO 1	.00300	.0350	4.77	434.93
4.8	94.08	10.0	2.0 TO 1	.00300	.0350	4.83	454.41
4.9	97.02	10.0	2.0 TO 1	.00300	.0350	4.88	473.46
5.0	100.00	10.0	2.0 TO 1	.00300	.0350	4.94	494.00
5.1	103.02	10.0	2.0 TO 1	.00300	.0350	4.99	514.07
5.2	106.08	10.0	2.0 TO 1	.00300	.0350	5.04	534.64
5.3	109.18	10.0	2.0 TO 1	.00300	.0350	5.09	555.73
5.4	112.32	10.0	2.0 TO 1	.00300	.0350	5.15	578.45
5.5	115.50	10.0	2.0 TO 1	.00300	.0350	5.20	600.60
5.6	118.72	10.0	2.0 TO 1	.00300	.0350	5.25	623.28
5.7	121.98	10.0	2.0 TO 1	.00300	.0350	5.30	646.49

DITCH HYDRAULICS PROGRAM

DEPTH FT.	AREA SQ.FT.	B.W. FT.	AVE. S.S.	CHAN. GRADE	MAN. N	VEL. F.P.S.	FLOW CFS
5.8	125.28	10.0	2.0 TO 1	.00300	.0350	5.35	670.25
5.9	128.62	10.0	2.0 TO 1	.00300	.0350	5.39	693.26
6.0	132.00	10.0	2.0 TO 1	.00300	.0350	5.44	718.08
6.1	135.42	10.0	2.0 TO 1	.00300	.0350	5.50	744.81
6.2	138.88	10.0	2.0 TO 1	.00300	.0350	5.55	770.78
6.3	142.38	10.0	2.0 TO 1	.00300	.0350	5.60	797.33
6.4	145.92	10.0	2.0 TO 1	.00300	.0350	5.65	824.45
6.5	149.50	10.0	2.0 TO 1	.00300	.0350	5.70	852.15
6.6	153.12	10.0	2.0 TO 1	.00300	.0350	5.73	877.38
6.7	156.78	10.0	2.0 TO 1	.00300	.0350	5.78	906.19
6.8	160.48	10.0	2.0 TO 1	.00300	.0350	5.83	935.60
6.9	164.22	10.0	2.0 TO 1	.00300	.0350	5.88	965.61
7.0	168.00	10.0	2.0 TO 1	.00300	.0350	5.93	996.24

OUTLET CHANNEL HYDRAULICS

COOPERATOR OR PROJECT= WR GRACE DAM

DESCRIPTION= OUTLET CHANNEL

CALCULATED BY LLB

12-13-1991

DITCH HYDRAULICS PROGRAM

DEPTH FT.	AREA SQ.FT.	B.W. FT.	AVE. S.S.	CHAN. GRADE	MAN. N	VEL. F.P.S.	FLOW CFS
1.0	11.50	10.0	1.5 TO 1	.04000	.0400	6.61	76.01
1.1	12.82	10.0	1.5 TO 1	.04000	.0400	7.03	90.12
1.2	14.16	10.0	1.5 TO 1	.04000	.0400	7.38	104.50
1.3	15.54	10.0	1.5 TO 1	.04000	.0400	7.72	119.97
1.4	16.94	10.0	1.5 TO 1	.04000	.0400	8.06	136.54
1.5	18.38	10.0	1.5 TO 1	.04000	.0400	8.34	153.29
1.6	19.84	10.0	1.5 TO 1	.04000	.0400	8.67	172.01
1.7	21.34	10.0	1.5 TO 1	.04000	.0400	8.94	190.78
1.8	22.86	10.0	1.5 TO 1	.04000	.0400	9.26	211.68
1.9	24.42	10.0	1.5 TO 1	.04000	.0400	9.52	232.48
2.0	26.00	10.0	1.5 TO 1	.04000	.0400	9.78	254.28
2.1	27.62	10.0	1.5 TO 1	.04000	.0400	10.04	277.30
2.2	29.26	10.0	1.5 TO 1	.04000	.0400	10.29	301.09
2.3	30.93	10.0	1.5 TO 1	.04000	.0400	10.54	326.00
2.4	32.64	10.0	1.5 TO 1	.04000	.0400	10.79	352.19
2.5	34.37	10.0	1.5 TO 1	.04000	.0400	11.04	379.44
2.6	36.14	10.0	1.5 TO 1	.04000	.0400	11.28	407.66
2.7	37.93	10.0	1.5 TO 1	.04000	.0400	11.48	435.44
2.8	39.76	10.0	1.5 TO 1	.04000	.0400	11.72	465.99
2.9	41.61	10.0	1.5 TO 1	.04000	.0400	11.91	495.58

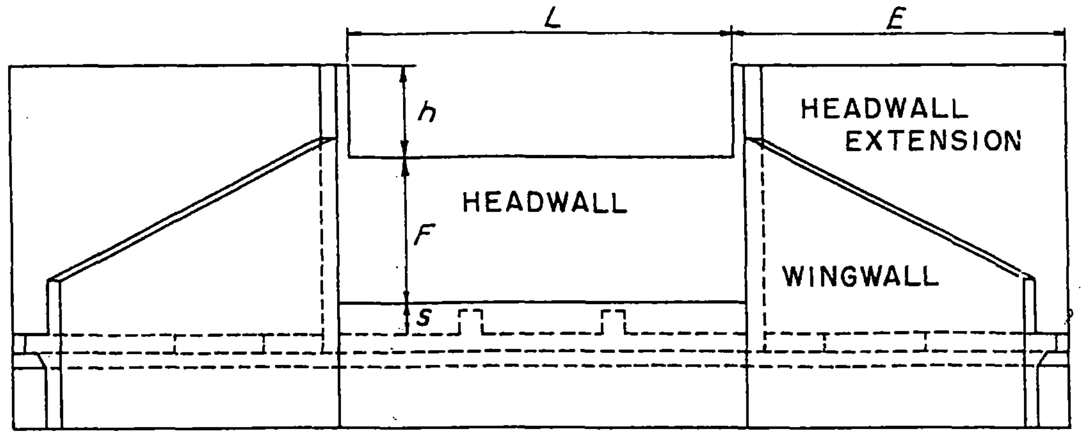
DITCH HYDRAULICS PROGRAM

DEPTH FT.	AREA SQ.FT.	B.W. FT.	AVE. S.S.	CHAN. GRADE	MAN. N	VEL. F.P.S.	FLOW CFS
3.0	43.50	10.0	1.5 TO 1	.04000	.0400	12.15	528.52
3.1	45.41	10.0	1.5 TO 1	.04000	.0400	12.34	560.36
3.2	47.36	10.0	1.5 TO 1	.04000	.0400	12.57	595.32
3.3	49.33	10.0	1.5 TO 1	.04000	.0400	12.76	629.45
3.4	51.34	10.0	1.5 TO 1	.04000	.0400	12.99	666.91
3.5	53.37	10.0	1.5 TO 1	.04000	.0400	13.17	702.88
3.6	55.44	10.0	1.5 TO 1	.04000	.0400	13.36	740.68
3.7	57.53	10.0	1.5 TO 1	.04000	.0400	13.54	778.96
3.8	59.66	10.0	1.5 TO 1	.04000	.0400	13.76	820.92
3.9	61.81	10.0	1.5 TO 1	.04000	.0400	13.94	861.63
4.0	64.00	10.0	1.5 TO 1	.04000	.0400	14.13	904.32
4.1	66.21	10.0	1.5 TO 1	.04000	.0400	14.30	946.80
4.2	68.46	10.0	1.5 TO 1	.04000	.0400	14.48	991.30
4.3	70.73	10.0	1.5 TO 1	.04000	.0400	14.66	1036.90
4.4	73.04	10.0	1.5 TO 1	.04000	.0400	14.84	1083.91
4.5	75.37	10.0	1.5 TO 1	.04000	.0400	15.01	1131.30
4.6	77.74	10.0	1.5 TO 1	.04000	.0400	15.18	1180.09
4.7	80.13	10.0	1.5 TO 1	.04000	.0400	15.36	1230.80
4.8	82.56	10.0	1.5 TO 1	.04000	.0400	15.53	1282.16
4.9	85.01	10.0	1.5 TO 1	.04000	.0400	15.70	1334.66
5.0	87.50	10.0	1.5 TO 1	.04000	.0400	15.87	1388.63

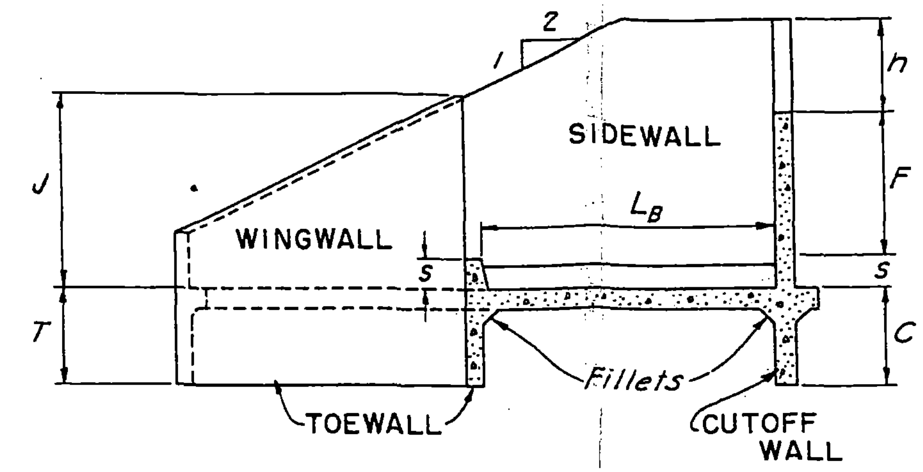
APPENDIX E

STANDARD DRAWING - SCS DROP STRUCTURE

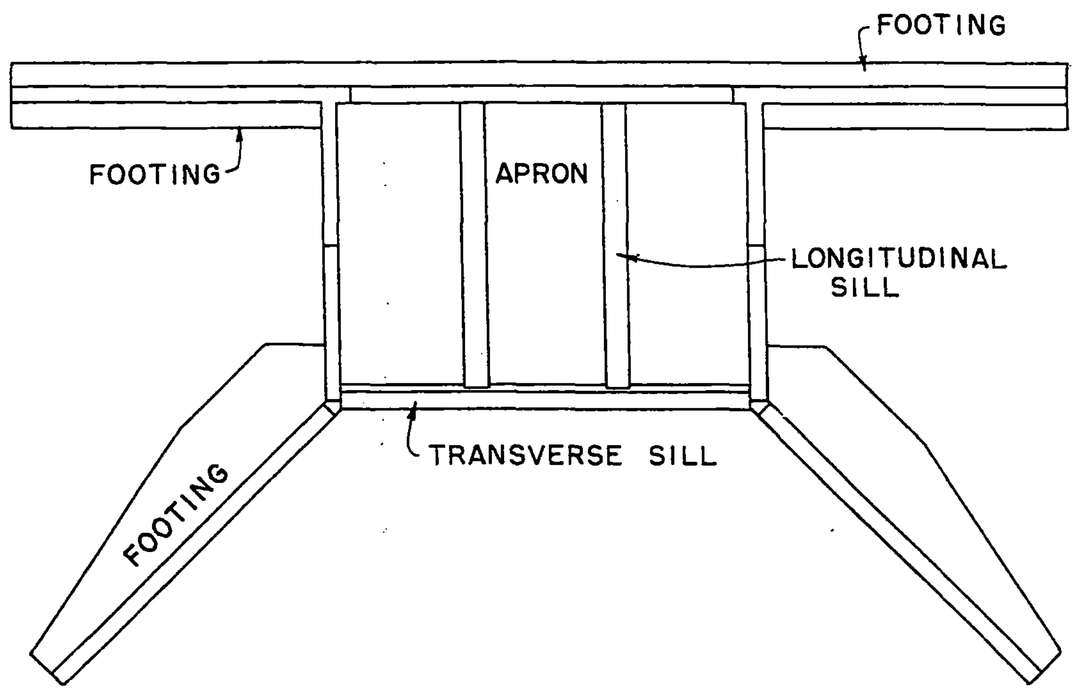
DROP SPILLWAYS: NOMENCLATURE AND SYMBOLS OF DROP SPILLWAY



DOWNSTREAM ELEVATION



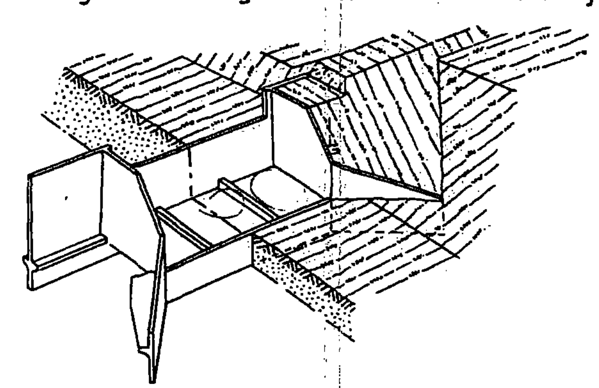
SECTION ON CENTER LINE



PLAN

SYMBOLS

- L = Length of weir.
- h = Depth of weir.
- F = Drop through spillway from crest of weir to top of transverse sill.
- s = Height of transverse sill.
- L_B = Length of apron.
- T = Depth of toewall below top of apron.
- C = Depth of cutoff wall below top of apron.
- d_c = Critical depth of weir.
- E = Length of headwall extension.
- J = Height of wingwall and sidewall at junction.



PERSPECTIVE VIEW

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING STANDARDS UNIT	STANDARD DWG. NO. ES-63 SHEET 1 OF 1 DATE 1-26-52
Rev. 12-14-53		

LSB 66 00129